ADAPTATION OF PRESTRESSED CONCRETE TO MODULAR GIRDER BRIDGE DESIGN FOR ADVANCED BASE CONSTRUCTION

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ADAPTATION OF PRESTRESSED CONCRETE
TO MODULAR GIRDER BRIDGE DESIGN
FOR ADVANCED BASE CONSTRUCTION

Submitted to the Faculty of Rensselaer Polytechnic Institute in partial fulfillment for the degree of Master of Science in Civil Engineering.

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and

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Troy, New York

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FOREWARD

In recent years prestressed concrete construction has attracted unusual attention abroad and in this country for the following reasons. It is in effect a new construction material or approach; it is more economical for certain structures; and it results in a considerable saving of materials. The Europeans and, very recently, the engineers in the United States have made great progress in a relatively short time in developing and using this essentially new material-prestressed concrete.

Relatively few linear prestressed structures have been designed and constructed in the United States to date, due to the high cost of installing the prestressed cables and the higher cost of complicated concrete forms. It is believed that these objections can be overcome for the most part by the mass production of standard precast prestressed units, and such an approach has recently been employed by several state highway commissions. This type of prestressed concrete construction is considered to be ideally suited for Naval advanced base construction of highway bridges; since it will result not only in a saving of material and critical shipping space, but will permit also construction of the bridges in a minimum length of time. Therefore, the design of a series of modular prestressed concrete bridges for advanced base construction has been undertaken as the subject of this thesis.

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The authors are indebted to Rear Admiral L. B. Combs, USN, ret., Head of the Department of Civil Engineering at Rensselaer Polytechnic Institute, and Dr. J. Sterling Kinney, Professor of Civil Engineering, also of Rensselaer Polytechnic Institute, for their advice and guidance in the preparation of this thesis. The authors also desire to express their appreciation to Professor Henry Trathen, Professor of Mechanics, Rensselaer Polytechnic Institute, and Mr. W. E. Dean, Engineer of Bridges, State Road Department of Florida for their suggestions.

ABSTRACT

The design of a series of precast prestressed concrete bridges for Naval advanced base construction is presented in this thesis. Girder type concrete bridges composed of modular prestressed girders of 40, 60, 80, 100 and 120 foot spans have been conceived, in so far as possible, to reduce the materials and construction time required to a minimum, consistent with good design practise and economy. The designs are slanted toward mass production precast methods of construction.

The modular girders have been proportioned in section to obtain an efficient use of the concrete and the high strength steel for prestressing, in accordance with varying dead load to live load ratios for the different span lengths and for the different construction phases. The method of approach has utilized the bridge deck slab as a contiguous portion of the prestressed tee shaped girders for the live load plus impact for H 20-S 16-44 loading conditions. Although the designs have been adapted for Naval advanced base construction, they are considered to be equally applicable to state or local highway commission projects. While the modular girders have been conceived principally for bridge construction, it is believed that they are adaptable, with only slight modification, to pier and wharf construction. The use of such girders for pier and wharf construction furnishes the added advantage of long life due to the pro-

tection against corrosion of salt water afforded by the compressed concrete.

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Part I INTRODUCTION

A A History of Prestressed Concrete

Prestressed concrete is the imposition of initial or preliminary stresses in a concrete structure, before the working or live loads are applied, in such a manner as to achieve a more favorable state of stress when the working loads come into action. The prestressing is accomplished by tensioning high strength steel wires or rods which results in initial compressive stresses in the desired portion of the concrete structure. Thus, tensile stresses in the concrete due to the live loads are completely or partially nullified or negated by the initial impressed compressive stresses. The two major categories of prestressed concrete are as follows:

1. Pretensioned or Hoyer System - The rods or wires are tensioned prior to pouring of the concrete.

2. Post-tensioned System - The wires or rods are tensioned subsequent to the pouring and curing of the concrete.

The latter method is generally more popular in the Unites States because it requires no extensive tensioning beds or frames, and the post-tensioned system has been selected

for this design thesis.

The fundamental theories of prestressed concrete are not new, having been conceived by Mr. P. H. Jackson of San Francisco as early as 1886; however, the application was not successful or practicable until the development of high strength steel wire was accomplished in the 1930's. Other early contributions were made by the German C. F. W. Doehring in 1888, the Austrian Wetstein, the American Mr. R. H. Dill in 1928 and the French Engineer E. Freyssinet in 1928. Mr. Freyssinet developed the new homogeneous material by bonding high strength steel wires to concrete and investigated the effects of creep, shrinkage and plastic flow. Mr. Hoyer of Germany perfected the pretensioning process in 1938 by using high strength piano wires in his structures. Professor G. Magnel of Belgium perfected a method of postensioning small diameter wires by the employment of "sandwich plates", and other tensioning methods were developed by Freyssinet and the American H. Shorer.

The development and application of prestressed concrete has been directly dependent on the progress in the manufacture of high strength steel wires and bars. Swedish engineers and metallurgists have developed wires having an ultimate tensile strength of 300,000 pounds per square inch. The Lee-McCall system, which was developed in England, employs relatively large steel bars having an ultimate strength of approximately 160,000 psi. and permits rapid

post-tensioning of the rods. Probably the most important single development is the "Roebling Method" of producing high strength prefabricated bridge cables, and these cables have an ultimate strength of approximately 250,000 psi. These new discoveriex in the steel industry have resulted in greater compatibility between the steel and concrete in the prestressed structures and have considerably reduced the construction time and cost.

The above discussion concerns primarily linear prestressed structures since the subject of this paper is a linear design, however, similar advances have been made in the field of circular pipes and tanks. The original concepts were applied by Mr. Hewitt, Mr. Crom and Professor Crepps in this country and by Mr. Freyssinet in France in the 1930's. The Preload Enterprises Incorporated of New York City has been the leader in the field of prestressed tank and pipe construction.

B <u>The Advantages of Prestressed Concrete Construction</u>

Prestressed concrete has attracted unusual attention abroad and in this country because it generally results in better structures; it effects a saving in critical materials in times of national emergency and conserves natural resources over a long period of time; and it results in more economical construction in certain types of structurds. The advantages of prestressed concrete are summarized as

follows[1]:

1. Prestressing makes concrete crackless, which is conducive to greater durability under severe conditions of exposure.

2. Prestressing makes it possible to use efficiently higher strength concrete of correspondingly better quality.

3. Prestressing makex it possible to use thin web concrete members of I and Tee sections, thereby obtaining the most effective distribution of material.

4. Prestressing minimizes deflection and reduces the depth of beams and girders and the thickness of slabs, thus affording greater under clearance.

5. Prestressing results in maximum rigidity under working loads and maximum flexibility under excessive overloads to give ample warning to impending failure.

6. Prestressing makes it possible to design each structure, specifically, to fit job requirements.

The lighter weight structure and shallow sections not only provide greater clearances but also cause an attendant reduction in foundation and pier costs. Pre-

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1 "Why Prestressed Concrete", by L.H. Corning; Proceedings of the First U. S. Conference on Prestressed Concrete, August, 1951.

stressing has increased the range of economical span lengths approximately as follows[2]:

Slab Bridges	30 to 80 feet
Simple Span Girders	60 to 150 feet
Continuous Girder Bridges	80 to 300 feet.
And Flat Prestressed Arches or Tied Girder Bridges	80 to 300 feet.

In general, as compared to ordinary reinforced concrete, prestressed concrete will result in the saving of 25% to 50% of the concrete and a saving of up to 75% of the steel required. Prestressed concrete has been recently successfully adapted to continuous, poured in place and precast girder bridges. In order to obtain continuity, additional cables or wires are located over the supports after the simple span precast girders have been placed.

The most apparent disadvantages to prestressed concrete are the relatively high cost of the high strength steel and the time and expense involved in the tensioning operation. Also, the concrete forms are generally more expensive for the intricate section shapes, but this disadvantage can be partially offset by mass production precasting methods.

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2 "General Design and Economic Considerations In the Planning of Prestressed Concrete Structures", by M. Fornerod, Proceedings of the First U.S. Conference on Prestressed concrete, August, 1951.

Another disadvantage for some structures is that the linear prestressed members cannot be efficiently reinforced to resist a reversal of stresses or loading.

Until recently, prestressed concrete construction could not compete economically with ordinary reinforced concrete construction for the reasons outlined above. However, by designing for precast prestressed construction methods, the highway departments of Florida, Pennsylvania and Massachusetts have been able to construct prestressed bridges more economically than ordinary reinforced structures.

C Objectives And Scope

The design of a series of precast prestressed concrete bridges for Naval advanced base construction is presented in this thesis. Girder type concrete bridges composed of modular prestressed girders and deck slab of 40, 60, 80, 100, and 120 foot spans have been conceived, in so far as possible, to reduce the materials and construction time required to a minimum, consistent with good design practice and economy. The designs are slanted toward mass production precast methods of construction.

In designing for Naval advanced base construction it is necessary to conceive the complete construction proceedures for rapid completion of field work and the design must be adapted accordingly. It is the objective of these designs to reduce the materials and shipping space required

to a minimum, since both are critical in times of a national emergency. Precast prestressed concrete modular units are considered ideally suited for these purposes and therefore have been selected. It is considered that it will be necessary to ship only the prestressing cables, nominal quantities of reinforcing bars for temperature steel and deck slab, cement, and standard steel forms to the advanced base site. Concrete aggregate can be obtained locally and the necessary heavy construction equipment will be required at the site for other construction projects as well as for the prestressed concrete bridges. This approach serves as the basis of the designs by the authors.

The objective of this thesis is not to develop a design for the construction of a single span deck girder bridge at a small advanced base, as such would be an uneconomical operation. It is intended that the designs and methods of construction proposed would be utilized at a large advanced base where a number of bridges would be required; such as the Guam, Okinawa, or Manus bases of World War II. For example, at the large Guam base some fourteen bridges and large culverts up to 90 foot span were required. After the combat phase of the operation is completed, and after the combat trails and roads are replaced during the base development phase, many bridges are required for the large bases and must be constructed in a minimum of time. Modern methods of advanced base construction require that the port-

able "Bailey" and other types of bridges be "rolled up" for reuse at another location. Thus a demand is created for easily constructed, semi-permenant bridges; such as the prestressed girder bridges presented in this paper. Advanced base construction methods and equipment impose certain detailed design requirements which are set forth under the topics of Design Criteria and Construction Procedures.

The design criteria and specifications are developed for advanced base construction but are tailored and equally applicable to continental highway requirements. It is the intent that the designs will be equally suitable for thruway commission, state highway department and local road commission projects that require the H 20-S 16-44[3] loading conditions.

It is further proposed in this design study to compare the prestressed structures to ordinary concrete structures for the same loading conditions and span lengths. Comparisons are made of the effect of the dead load to live load ratio for the different span lengths considered. The savings in material is the result of integral or contiguous design of the bridge slab and the prestressed precast girders will be discussed as well as the limitations of the designs.

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3 Standard Specifications for Highway Bridges, American Association of State Highway Officials, Fifth Edition, 1949

The adaptation of prestressed integral Tee and I beams to continuous structures is discussed.

It is believed by the authors that good useable designs are presented but also the additional value of this thesis is its worth as a stimulus for the use of prestressed concrete in bridges, especially the method advanced herein; and as a guide to span length and section selection by the individual designer.

II DESIGN THEORY

Since prestressed concrete members must satisfy several condition equations, the design of such members is generally a process of trial and error based largely on previous experience and judgement. To begin the discussion of the design theory, a system of symbols and notations is furnished as follows:

A. Symbols and Notations

A = Cross-sectional area of a beam.

A_s = Cross-sectional area of steel reinforcement.

α = angle of prestressing cable with the neutral horizontal axis.

b, B = width or breadth.

C, C_1, C_2 = Constants relating to the ratio of stresses.

C_{at} = calculated stresses in the top fiber due to loads applied subsequent to prestressing.

C_{dt} = calculated stresses in the top fiber due to loads existing before prestressing.

C_{ab} = calculated stresses in the bottom fiber due to loads applied after prestressing.

C_{db} = calculated stresses in the bottom fiber due to loads existing before prestressing.

C_t = permissable tensile stress in the concrete.

D = total depth of the beam or slab.

d = effective depth of the concrete beam or slab, - depth to tensile reinforcement.

E_c = Modulus of elasticity for concrete in compression.

E_s = modulus of elasticity for steel.

e = eccentricity from centroid or center of gravity.

e_t = eccentricity toward the top fibers.

e_b = eccentricity toward the bottom fibers.

f_c = allowable unit compressive stress in the extreme fiber of concrete.

f'_c = ultimate compressive stress of concrete per A.S.T.M. tests on 6 x 12 cylinders.

f_s = allowable unit stress in steel-tension or compression.

I = moment of inertia of a section about the neutral axis for bending.

j = ratio of lever arm of resisting couple to depth d.

k = ratio of depth of neutral axis to depth d.

K = $\frac{1}{2} f_c k j$ for concrete beams, a ratio.

l,L = Span length or distance.

M = bending moment.

M_a = bending moment due to loads applied after prestressing.

M_d = bending moment due to loads applied before prestressing.

$n = \frac{E_s}{E_c}$, ratio of modulus of elasticity of steel to concrete.

p = ratio of effective area of tension reinforcement to effective area of concrete = $\frac{A_s}{bd}$.

P = axial load or prestress force.

P_i = initial prestress force, prior to creep, shrinkage and plastic flow.

Q = statical moment of resistance about the neutral axis of the area of a section on one side of the neutral axis.

r = radius of gyration = $\sqrt{\frac{I}{A}}$.

v = shearing unit stress.

V = total shear.

w = uniformly distributed load per unit length.

x = a distance along the horizontal x axis.

y = a distance along the vertical y axis.

z = a distance along the third dimensional axis.

η = eta, proportion of prestress force remaining after creep, shrinkage and plastic flow have occured = 0.85

B. General Considerations and Formulae

The basic design theory employed by the authors is that presented by Professor Gustave Magnel in his book, "Prestressed Concrete."[4] Certain additional theoretical equations were developed by Dr. J. Sterling Kinney, Professor of Civil Engineering, Rensselaer Polytechnic Institute in connection with his course of lectures on Prestressed Concrete and have

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4 Magnel, G., Prestressed Concrete, Concrete Publications Ltd. 2nd Edition, 1950, London.

been used as a basis for design. The theory of ordinary reinforced concrete design as used in design calculations is not discussed, since the ordinary reinforced concrete design theory is generally known and accepted.

The general formula for the linear prestressed member subjected to an eccentric axial load, P, and a bending moment M has been given in many textbooks in various modifications:

$$f_c = \frac{P}{A} \pm \frac{Pey}{I} \pm \frac{My}{I}$$

Professor Magnel has applied this equation to four separate conditions of design. The prestressed concrete member subjected to bending only, in general, must resist a bending moment resulting from the dead load, a bending moment resulting from the live load, and a bending moment resulting from the prestressing force. Due to the dead load moment the stresses in the top and bottom fibers are:

$$C_{dt} = \frac{My_t}{I} = \frac{M}{S_t}$$

$$C_{db} = \frac{My_b}{I} = \frac{M}{S_b}$$

Due to the live load moment the stresses in the top and bottom fibers are:

$$C_{at} = \frac{My_b}{I} = \frac{M}{S_t}$$

$$C_{ab} = \frac{My_t}{I} = \frac{M}{S_b}$$

The above stresses combined with the effect of the prestressing force result in the final stress in the extreme fiber of the beam or member. The section of the beam at midspan or centerline must satisfy the following conditions:

(1) Initially upon application of the prestressing force and with the dead load of the beam effective, the allowable tensile stress must not be exceeded in the top fiber. This is expressed by the equation:

$$\frac{P_i}{A}\left(\frac{e y_t}{r^2} - 1\right) - C_{dt} \leqq C \qquad (1)$$

(2) Upon application of the added or live loads, the compressive stress in the top fiber must not exceed the permissible compressive stress. Creep in the steel and shrinkage and plastic flow in the concrete are assumed to have occurred.

When $e > \frac{r^2}{y_t}$,

$$-\eta \frac{P_i}{A}\left(\frac{e y_t}{r^2} - 1\right) + C_{dt} + C_{at} \leqq f_c \qquad (2)$$

When $e < \frac{r^2}{y_t}$

$$\frac{P_i}{A}\left(1 - \frac{e y_t}{r^2}\right) + C_{dt} + C_{at} \leqq f_c \qquad (2a)$$

(3) Initially upon application of the prestressing force, the compressive stress in the bottom fiber must not exceed the allowable.

$$\frac{P_i}{A}\left(1 + \frac{e\,y_b}{r^2}\right) - C_{db} \leqq f_c \tag{3}$$

(4) After the daad and live loads are acting, the tensile stress in the bottom fiber must not exceed the allowable.

$$-\frac{\eta P_i}{A}\left(1 + \frac{e\,y_b}{r^2}\right) + C_{db} + C_{ab} \leqq C_t \tag{4}$$

C. Determination of the Concrete Section and the Prestress Force

By combining equations three and four the following is obtained:

$$\frac{I}{y_b} \geqq \frac{M_a}{f_c + C_t - (1-\eta)\frac{P_i}{A}\left(1 + \frac{e\,y_b}{r^2}\right)} \tag{5}$$

By combining (1) and (2), similarly:

$$\frac{I}{y_t} \geqq \frac{M_a}{f_c + C_t - (1-\eta)\frac{P_i}{A}\left(\frac{e\,y_t}{r^2} - 1\right)} \tag{6}$$

Thus two of the four conditions are satisfied in each of equations (5) and (6). In most cases of design an upper limit can be determined fairly accurately for the terms.

$$(1-\eta)\frac{P_i}{A}\left(1 + \frac{e\,y_b}{r^2}\right) \text{ and } (1-\eta)\frac{P_i}{A}\left(\frac{e\,y_t}{r^2} - 1\right).$$

In a properly designed beam, $\frac{P_i}{A}$ is about $0.5\ f_c$, $\frac{e y_t}{r^2}$ and $\frac{e y_b}{r^2}$ are each generally equal to approximately 2.0, and η (eta) is normally assumed to be 0.85. This permits a general approximation of equations (5) and (6) as follows:

$$\frac{I}{y_b} = \frac{M_a}{0.775\ f_c + C_t} \tag{8}$$

$$\frac{I}{y_t} = \frac{M_a}{0.925\ f_c + C_t} \tag{9}$$

With these formulae the section modulus and beam dimensions of a trial beam can be computed.

The section modulus and moment of inertia of irregular I and Tee sections may be computed from the following equations:

Moment of interia about the base:

$$I_b = \sum I_{cg} + A_y{}^2 \tag{10}$$

Distance to neutral axis:

$$Y_b = \sum \frac{A_y}{A} \tag{11}$$

Moment of inertia about the neutral axis:

$$I_{cg} = I_B - Y_b{}^2 \sum A \tag{12}$$

Section moduli :

$$S_b = \frac{I}{y_b} \tag{13}$$

$$S_t = \frac{I}{y_t} \tag{14}$$

The mimimum eccentricity of the prestress steel to satisfy the condition equations may be obtained by combining equations (2) and (3). By division and substitution:

$$e = \frac{\left(C_1 + 1\right) r^2}{y_t - C_1 y_b} \tag{15}$$

where $$C_1 = \frac{C_{dt} + C_{at} - f_c}{\eta (C_{db} + f_c)} \tag{16}$$

The maximum eccentricity of the prestress steel permissible to satisfy the condition equations may be obtained by combining equations (1) and (4)

$$e = \frac{(C_2 + 1) r^2}{y_t - C_2 y_b} \tag{17}$$

where,

$$C_2 = \frac{(C_t + C_{dt}) n}{(C_{db} + C_{ab} - C_t)} \tag{18}$$

By re-arranging equation (4), the necessary initial prestressing force may be computed.

$$P_i = \frac{(C_{db} + C_{ab} - C_t) A}{(1 + \frac{e y_b}{r^2}) n} \tag{19}$$

The above condition equations were derived for the maximum bending moments occurring at the mid span point for

a simply supported beam with a uniform load or concentrated load at the midspan. These basic equations may be extended to apply throughout the length of the beam in order to assure that the compressive stresses or tensile stresses in the concrete do not exceed the allowable amounts. The cable trajectory must be so adjusted throughout the length of the beam that the eccentricity is neither too great or too small. Quite often a parabolic curve is employed for the cable trajectory to obtain the proper excentricity. Frequently, curves are plotted showing the effects of dead load, live load and the prestressing force throughout the length of the beam.

The state of stress at the end of the beam is of particular importance, since for a simple beam the end moments are equal to zero. The eccentricity of the prestressing cable or wires must fall within the "core" of the section to prevent excessive tension in the top fiber of the beam at the time the initial prestressing force is applied. This particular condition is satisfied by equation (1), when the moment due to the weight of the beam is equal to zero.

In the design of this series of prestressed concrete bridges, the bridge deck slab and its supporting members have been keyed and bonded together to act as a contiguous or integrated tee beam upon application of the live load plus impact. The precast girders(supporting members of the bridge deck slab) are designed to carry the forms and the concrete

slab as an added load and must satisfy the above condition equations. In addition the composite girders, consisting of the bridge longitudinal girders and proportional sections of the deck slab, must also satisfy these condition equations. Since the girder goes thru a transition to the composite girder section during the curing of the concrete the properties must be calculated and proportioned in accordance with these same condition equations to satisfy the criteria of all conditions of loading by trial and error, judgement and experience. There is no direct solution for a proper beam section, eccentricity of the prestressing force, or the magnitude of the prestressing force.

D. <u>Shear and the Principal Tensile Stress</u>

Another important consideration in the design theory is the shear or principal tensile stress in the concrete at the point of greatest shear, usually at the end of a simply supported beam. Prestressed members generally have low values of principal tensile stress, permitting the use of the more effective and economical Tee and I sections. There are two factors which tend to reduce the principal tensile stress in a prestressed linear member. First, the prestressing axial load or force causes a compressive stress throughout the section; and, secondly, when curved or variably displaced cables or wires are employed, the cables or wires exert a vertical component of force upwards which cancels out a portion of the dead load and live load shears.

The shearing stress in a homogeneous or uncracked concrete section is a maximum at the centroid and may be computed by the following equation presented in many text-books:

$$v = \frac{V\ Q}{b\ I} \tag{20}$$

The V or external shear on the section is the sum of the dead and live load shears plus impact, minus the vertical prestress force. It is possible that the prestress force minus the dead load shear is greater than V above although this is not normally the case. The verical prestress force may be computed:

$$V^{c} = \eta \therefore P_i \sin \alpha \tag{21}$$

Where, α is the angle that the prestressing force or cable makes with the neutral horizontal axis. Parabolic cable trajectories are employed in this design of the girders, since such a trajectory satisfies the basic condition equations by complementing the dead and live load moment curves through-out the length of the girder. For a parabolic cable trajectory:

$$x^2 = 2 \rho\ y \tag{22}$$

$$\tan \alpha = \frac{x}{\rho} \tag{23}$$

From which the sine of the angle may be computed and substituted in equation (16). The resulting end shear is equal to the sum of the vertical forces acting on one side of the section or:

$$V = V_{DL} + V_{LL} - V' \tag{24}$$

The combined or biaxial stress system existing at the neutral axis of the girder results in a principal tensile stress that may be expressed by:

$$p_t = \sqrt{v^2 + \left(\frac{C_x}{2}\right)^2} - \frac{C_x}{2} \tag{25}$$

The computed principal tensile stress cannot exceed the allowable for the concrete employed.

The shearing stress at the juction of the bridge deck slab and the concrete prestressed girder may be computed by equation (15). In this case, Q is equal to the first or statical moment of the section or area above the point of juction about the neutral axis. By ordinary reinforced concrete design procedure the necessary keys and stirrups may be designed.

E. The Cracking Factor

The Cracking Factor is a ratio of the live load moment necessary to cause the first crack in the concrete at the bottom fiber, compared to the design live load moment. Prestressed members are normally designed as crackless structures or, expressed in a different way, the allowable concrete tensile stresses are held to such a value as to assure that no cracks occur under the design loads. The cracking factor is computed by a modification of equation (4) which is equated to the modulus of rupture of the concrete.

$$\frac{n P_i}{A}\left(1 + \frac{e y_b}{r^2}\right) + C_{db} + CF. (C_{ab}) = MR \qquad (26)$$

Tensile cracks in prestressed concrete do not have the same significance as in conventional reinforced concrete. If the stress in the prestressing steel does not result in permanent strain, the cracks will close up and the section will again act as a homogeneous section. In conventional reinforced concrete design, the cracks first appear before full live load is reached, and once cracked, the concrete will rarely return to a condition of homogenuity.

F. The Ultimate Factor

The girder is investigated for its ultimate load carrying capacity in accordance with the theory of ultimate design of reinforced concrete. The Ultimate Factor is expressed as a ratio of the live load moment plus impact at the time of failure of the girder.

$$M_u = 1 M_{DL} + (U.F.) M_{LL} \qquad (27)$$

The design theory is based on the fact that the failure occurs when the high strength prestressed steel undergoes a relatively large permanent strain as it nears its ultimate capacity causing a relatively large plastic strain in the concrete in the extreme top fiber of the beam. Failure occurs abruptly in the concrete at the top fiber as a result of the excessive concrete strain or more accurately, prior permanent strain in the prestress steel. The stress in the top fiber is not

considered to vary linearly due to the plastic flow of the concrete, but is considered to reach the ultimate strength of the concrete over a small rectangular area at the top fiber of the beam. The failure is considered to occur in an advantageous manner since up to the design load the deflection is small and when the design load is exceeded the deflections are large giving warning of impending failure. As an approximate solution of the ultimate strength of the beam, the distance from the top fiber to the centroid of concrete resistance may be computed:

$$d_1 = \frac{.8\ f_s\ A_s}{2\ f_c'\ b} \tag{28}$$

The effective resisting depth for the steel:

$$jd = e + y_t - d_1 \tag{29}$$

Then the resulting resisting moment at ultimate strength or failure may be dxpressed

$$M_u = .8\ A_s\ f_s\ jd \tag{30}$$

The Ultimate Factor computation gives an indication of the safety factor of the design accomplished in accordance with the theory outlined in previous paragraphs.

G. End Block Stresses

In order to provide a transfer of the prestress force from the steel wires or cables to the concrete at the end of the beam without excessive stress concentrations, bearing plates or large washers are used. The design of these end

bearing devices depends upon the type of wire or cable employed for prestressing and well known structural steel and concrete formulae are employed.

To further distribute the load imposed by the prestress cables or wires and the end bearing plate of the beam, end blocks of larger rectangular cross section are normally employed and stirrups provided to assure cracking of the concrete does not occur.

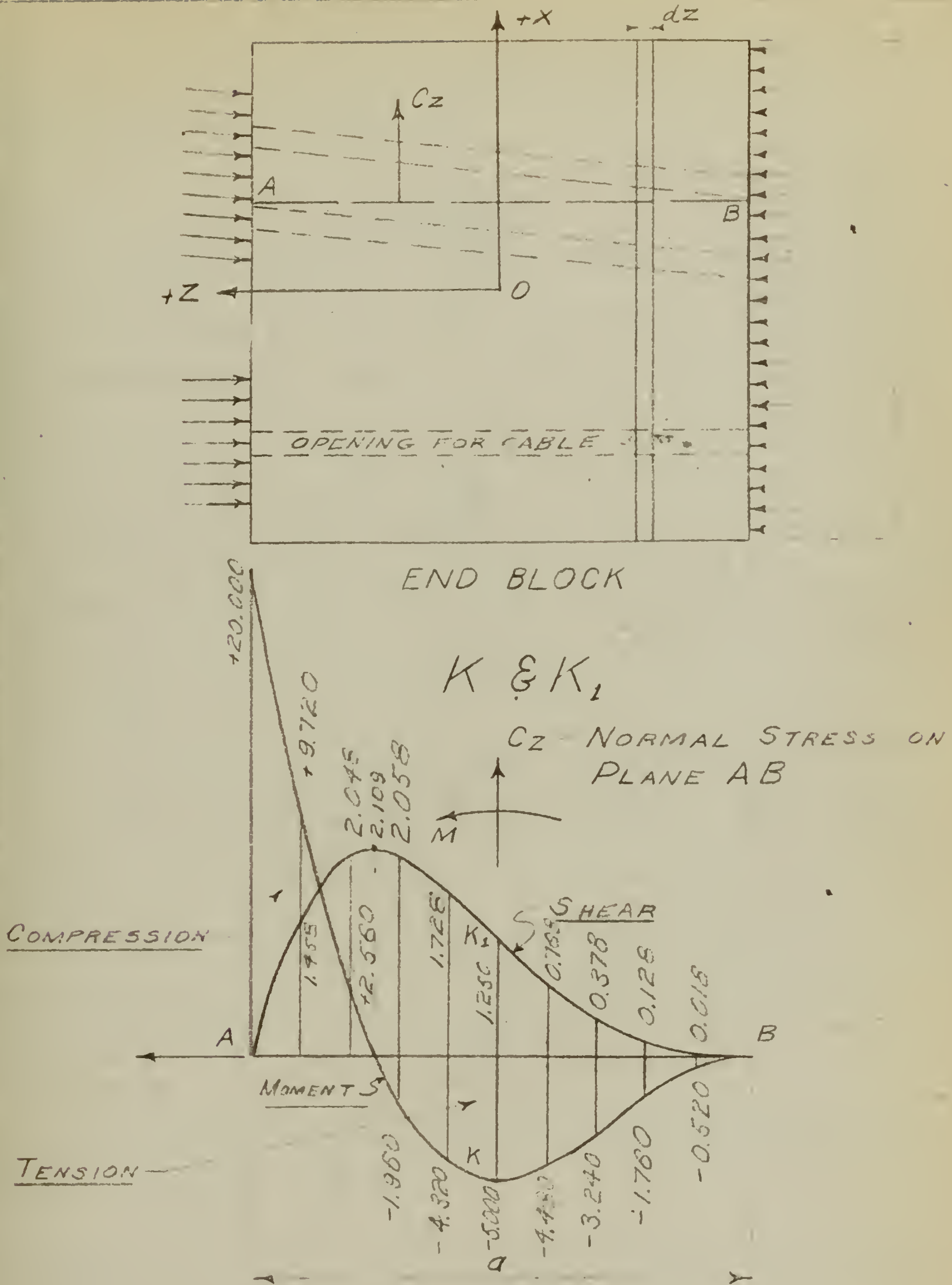
The principal tensile stress existing in the end block should be investigated in order to properly select the steel reinforcement required. As shown in Figure 1, the length of the end block is assumed to be equal to the depth of the beam and the stresses are assumed to vary in accordance with St. Vernant's Principle. Any horizontal plane such as A-B is considered to be acted upon by normal forces; a bending moment M and a shearing force S. The normal stress resulting from the end block bending moment varies in accordance with a third degree polynomial and the unit shearing stress varies in accordance with a fourth degree polynomial as follows:

$$C_z = \frac{5M}{ba^2}\left(-1 + \frac{12z^2}{a^2} + \frac{16z^3}{a^3}\right) = \frac{K \quad M}{ba^2} \qquad (31)$$

$$v = \frac{5\,S}{ba}\left(\frac{1}{4} + \frac{z}{a} \quad \frac{4z^3}{a^3} \quad \frac{4z^4}{a^4}\right) = K_1 \frac{S}{ba} \qquad (32)$$

Then for the biaxial stress sytem the principal tensile stress may be computed:

$$p_t = C_{\varphi} + C_z \pm \sqrt{v^2 + \left(\frac{C_x - C_z}{2}\right)^2} \qquad (33)$$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Cz NORMAL STRESS ON PLANE AB
M
+20.000
+9.720
2.045
2.109
2.058
1.455
+2.560
1.726
K1
1.250
0.789
0.378
0.126
0.018
SHEAR
COMPRESSION
A
B
MOMENTS
TENSION
K
-1.960
-4.320
-5.000
-3.240
-1.760
-0.520
a

FIGURE NO. 1

The above analysis of the end block stresses is presented in greater detail by Professor Magnel in his book, "Prestressed Concrete."[5] Values of the constants K and K_1 along the length "a" are also shown in Figure 1.

H. Deflections

The linear prestressed concrete members utilized in bridge structures generally have the property of relatively small deflections if curved or variably displaced prestressing wires or cables are employed. The total deflection is the algebraic sums of the deflection due to dead load, live loads the prestressing cable effect. Since the curved cables cause an upward vertical deflection, this is an additional advantage accruing to the prestressed concrete girder design. The prestressing force also assures that the beam or member is axially in compression and that no cracks will occur under design loads. This permits the utilization of the entire homogeneous concrete section to resist the loads, up to the cracking load of the beam or girder; or stated differently, the moment of inertia of the gross concrete section is effective in resisting deflections.

The deflections resulting from uniform loads may be computed from the equation:

5 Magnel, G., Prestressed Concrete, Concrete Publications Ltd., 2nd Edition, 1950, London

$$\Delta = \frac{5}{384} \quad \frac{w\ l^4}{E_c\ I} \tag{34}$$

By the method of Virtual Work, deflections may be computed for the concentrated wheel loads by the formula:

$$\Delta = \int_0^{\ell} \frac{M\ m\ dx}{E\ I} \tag{35}$$

The positive deflection upwards varies in accordance with the curve of the cable trajectory. For a parabolic cable trajectectory as has been used in the design of this series of bridges the deflections were computed in accordance with the method presented by Professor J. S. Kinney in his lectures on Prestressed Concrete. The general expression is derived by the means of Virtual Works.

$$\Delta = \int_0^{\ell} \frac{M_p\ \ m\ dx}{E\ I} \tag{36}$$

Where M equals the moment due to the prestressing force or

$$M_p \quad = \quad P \cos\alpha\ (e + y) \tag{37}$$

And y equals the distance from the neutral axis to the center of gravity of the cables at the end of the beam. For a parabolic cable trajectory;

$$\frac{\partial e}{\partial x} = \tan\alpha \qquad \cos\alpha = \frac{1}{\sqrt{1 + (\frac{\partial e}{\partial x})^2}}$$

$$e = A x^2 + B x + C \tag{38}$$

and solving the constants A, B and C by conditions of static equilibrium known to exist:

$$\Delta \mathrm{C\!\!\!L} = \frac{P_{e_o} L^2}{EI} \left[\frac{5}{48} - \frac{1}{10} \left(\frac{e_o}{L} \right)^2 \right] +$$

$$\frac{P_y L^2}{EI} \left[\frac{1}{8} - \frac{1}{6} \left(\frac{e_o}{L} \right)^2 \right] \tag{39}$$

In the above equation e_o equals the maximum cable eccentricity at the midspan point. Since Cos α is very nearly one, a close approximation of the deflection may be expressed:

$$\Delta \mathrm{C\!\!\!L} = \frac{5}{48} \quad \frac{P_{e_o} L^2}{EI} + \frac{P_y L^2}{8EI}. \tag{40}$$

In summary, the design of the prestressed, precast girders and the composite girders for the series of bridges presented in this thesis was accomplished in accordance with the condition equations enumerated in the foregoing paragraphs. In addition, the remainder of the concrete structure has been analized and designed in accordance with accepted ordinary reinforced concrete design procedures.

III DESIGN CRITERIA AND SPECIFICATIONS

The adaptation of precast, prestressed concrete for the construction of advanced base bridges imposes some rather rigid or severe requirements upon both the designer and the builder in the field. The structures must be capable of being erected in a short period of time, they must be capable of sustaining the heavy loads imposed by military traffic, and they must be sufficiently standardized and simplified to be built by military construction forces. The military forces should be

able to construct these bridges with the construction equipment normally available at advanced bases. A further requirement is that the materials shall be of low weight and bulk to conserve critical shipping space. If possible, the standard bridge girders should be suitable for other types of structures at the advanced base. With these objectives in mind, the authors have prepared the design criteria and specifications that are presented in this section of the thesis. It is to be noted that a majority of the basic requirements enumerated result in construction enconomy, thus assuring that the designs are equally applicable to state highway or thruway commission construction projects.

A. General Specifications

The series of bridges is to be of the deck girder type, as shown in Figure 2, with precast, prestressed girders. The spans are to be simply supported with theoretical span lengths center to center of bearing of 40, 60, 80, 100 and 120 feet. The bridge slab and deck girders are to act as an integral or contiguous concrete structure to resist the live load plus impact. The girders shall be of I section and designed to carry the forms, deck slab and construction loads as shown in Figures 1 and 2. The bridge deck slab and guard rail are typical of ordinary reinforced concrete construction as shown in figure (3) and this provides a two lane roadway of 26 feet in width. Although the two lane

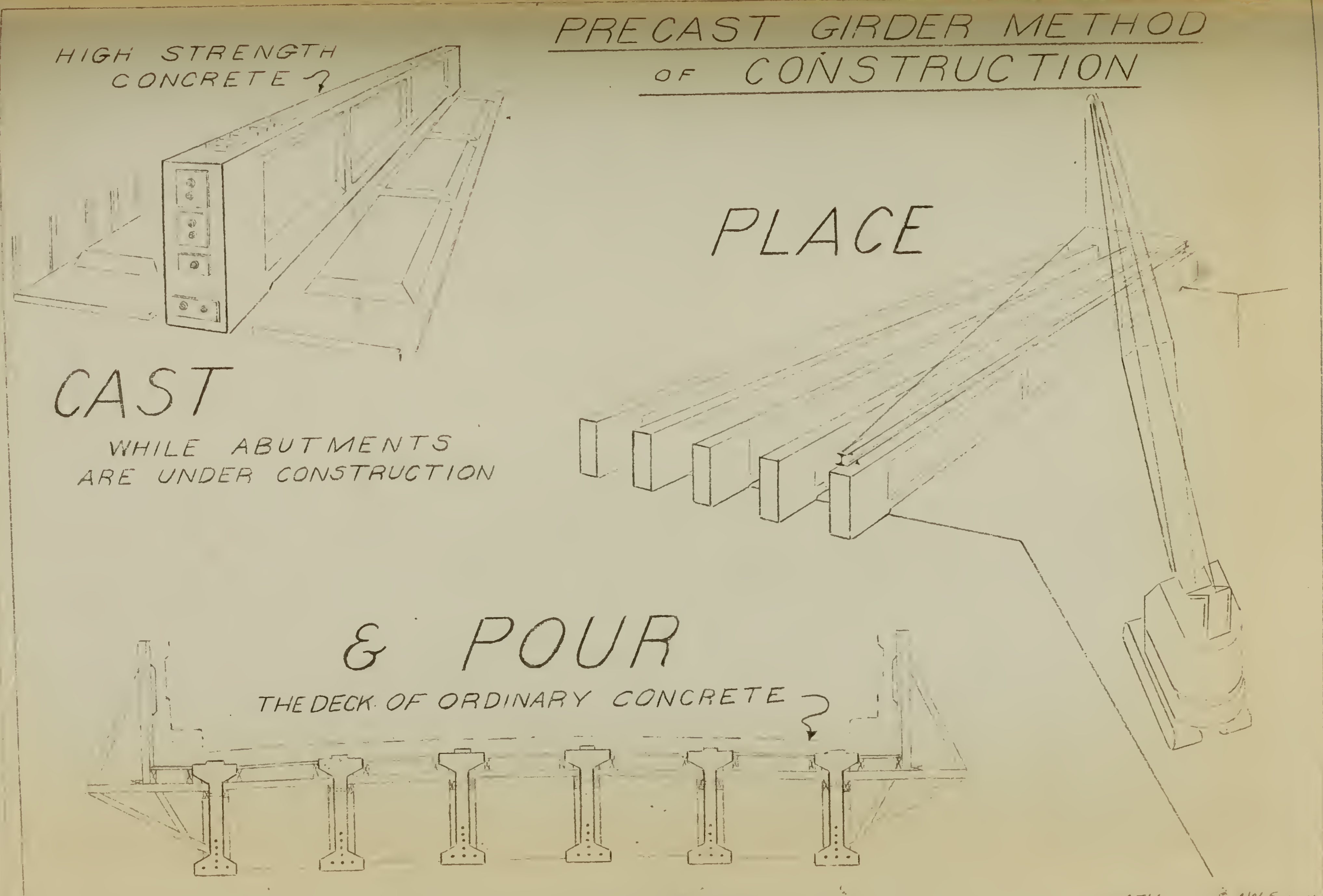
PRECAST GIRDER METHOD
OF CONSTRUCTION
HIGH STRENGTH
CONCRETE
CAST
WHILE ABUTMENTS
ARE UNDER CONSTRUCTION
PLACE
& POUR
THE DECK OF ORDINARY CONCRETE
AFMEEKS & AW SNOW

FIGURE No. 2

COMPARISON OF PRECAST GIRDERS ON A 20 FT. SPAN MODULE

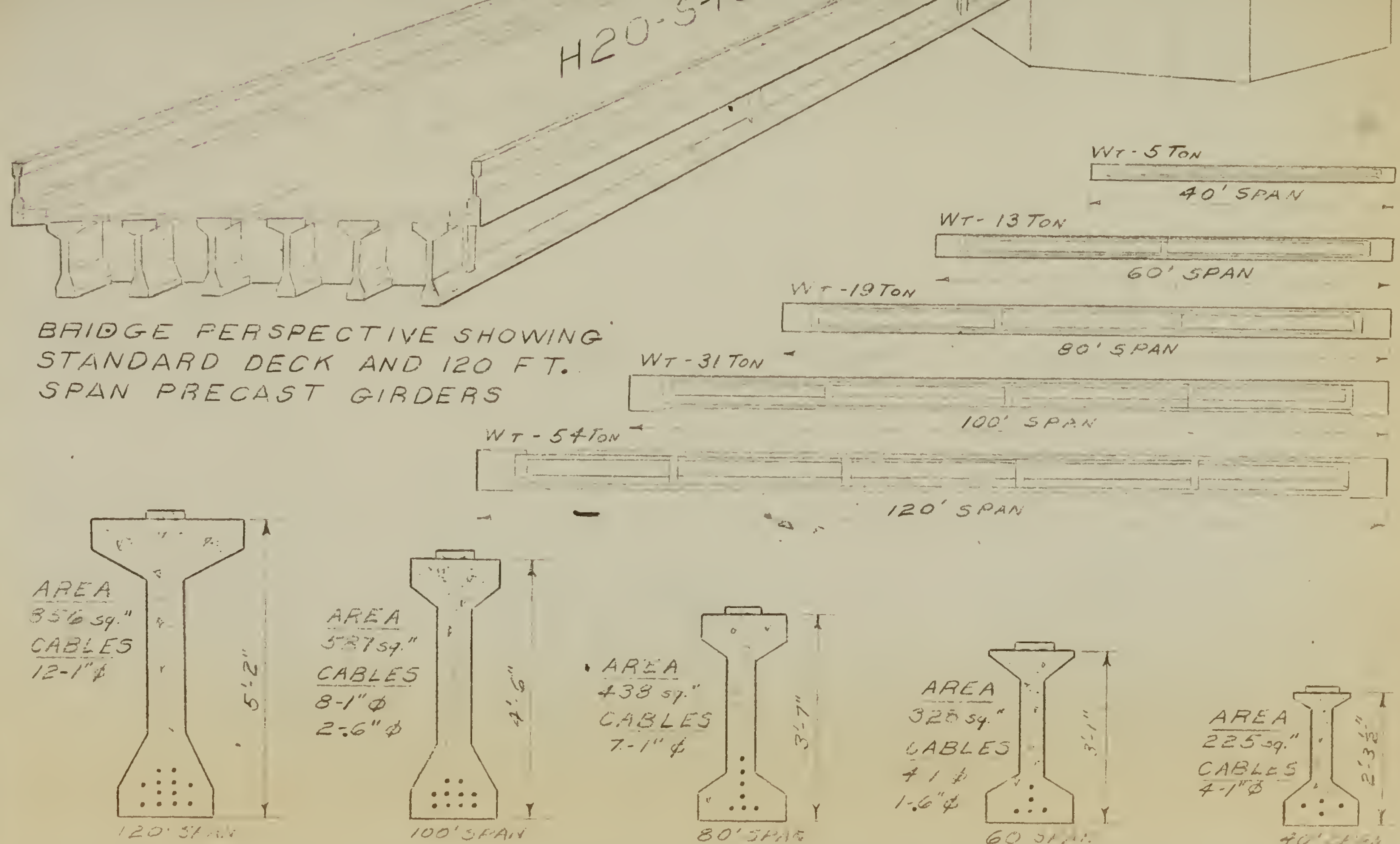

FIGURE No 3

roadway is presented in this design, the five foot spacing of girders will permit expansion to three or four lane width by the addition of girders in multiples of three. The overall width of the two lane structure is 32 feet with two 2 foot wide safety curbs. The general specifications for design are the Standard Specifications for Highway Bridges, American Association of State Highway Officials.[6] The construction materials are to conform to the A.A.S.H.O. Specifications and the American Society for Testing Materials Specifications.

B. Loads

The structures are to be proportioned for dead load, live load and impact or dynamic effect of the live load, all as described in Section 3.2 of the A.A.S.H.O. Specifications.[6] The design live load is to be H 20 - S16 - 44 and application of this loading shall be such as to produce the maximum stress. The standard truck or lane loads, whichever govern is assumed to be distributed over two and one-half of the prestressed concrete girders and the spacing of the girders is five feet center to center.

6. Standard Specifications for Highway Bridges, "American Association of State Highway Officials, Published by the Association, Washington, D. C., 1949

The impact factor for the dynamic effect of the live load shall be determined by the formula:

$$I = \frac{50}{L + 125}$$

as further defined in Section 3.2.12 of the A.A.S.H.O. Specification.

The distribution of the wheel loads to the slab shall be computed in accordance with Section 3.3.2 of the A.A.S.H.O. Specifications. The width of slab, E, over which the wheel load is distributed is calculated by the equations:

$$E = .6S + 2.5$$

The bending moment in the slab is computed by the formula:

$$M = \pm \frac{.2 \; P_1 \; S}{E}$$

where P_1 equals the load on one wheel of a single axle.

The I section, prestressed and precast girders shall be designed to carry all the span dead load, form loads and construction loads. The forms and construction loads have been assumed to be 0.5 times the dead load of the slab or 44 pounds per square foot.

C. Diaphrams

For Tee beams or the prestressed girders with spans of over forty feet in length, diaphrams or spreaders shall be placed between the beams at the middle or third points. This requirement is similar to the A.A.S.H.O. Specification for diaphrams employed with ordinary reinforced concrete

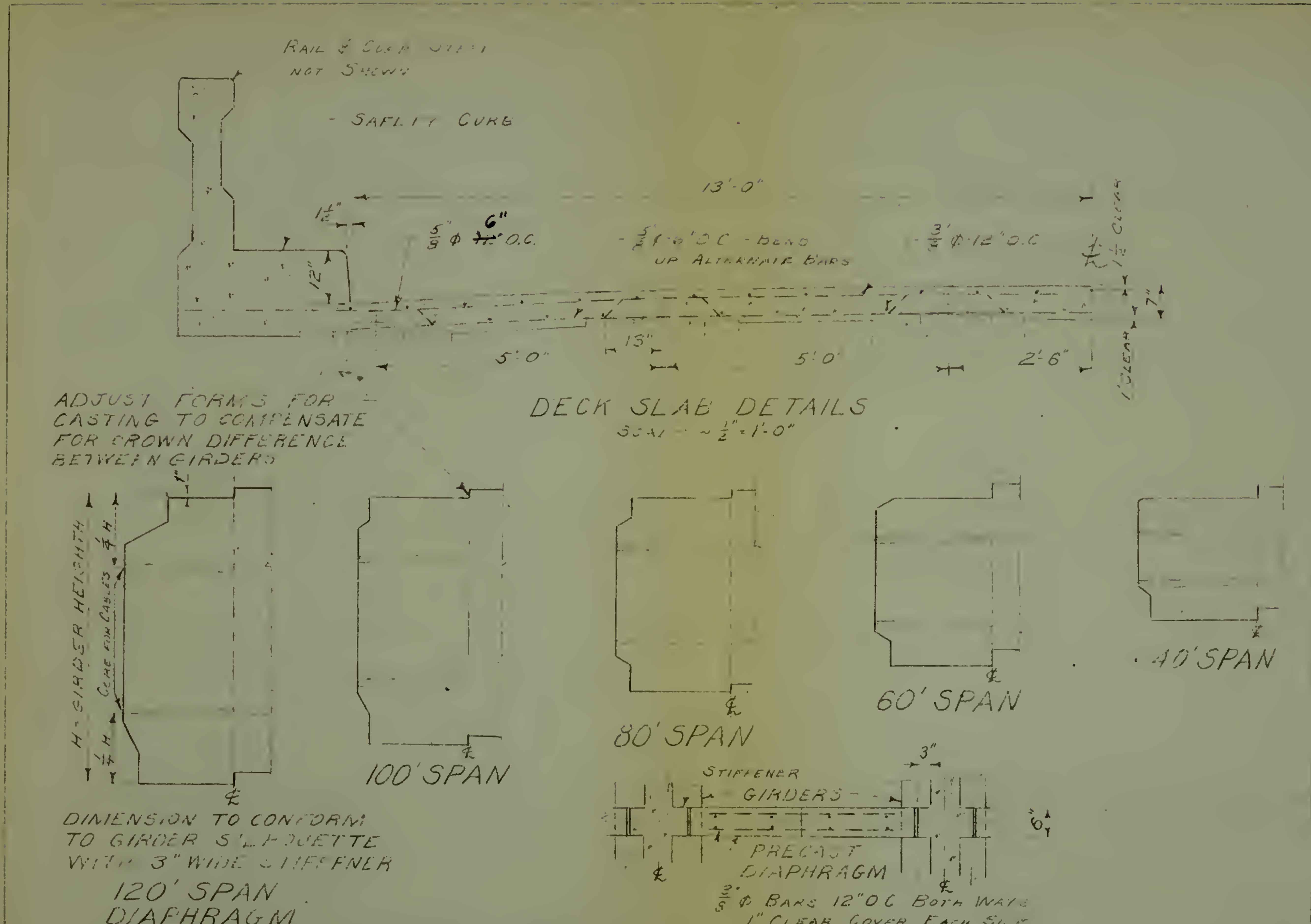

FIGURE No. 4 STANDARD DECK AND DIAPHRAGM DETAILS

Allowable principal tensile stress

for a biaxial stress system: .25 x modulus of

Rupture = .25 = 600 = 150 psi.

Allowable shear: v_c = .03 f'_c = 150 psi, without webb

steel.

Allowable bond strength: u = 0.075 f'_c = 375 psi.

Punching or pure Shear = 0.25 Ultimate Shearing Strength

= 0.25 x 2000 = 500 psi.

Bearing:

Full Area: .25 f'_c = 1,250 psi.

Less than half area: 0.375 f'_c = 1,875 psi.

Cable bearing plates: .5 f'_c = 2,5000 psi.

Modulus of Elasticity: E_c = 3,000,000 psi

for computing deflections.

Ordinary design: E_c = 1,000 f'_c = 5,000,000 psi.

and n = 6

The design of the concrete mix will depend upon the local aggregate available, but a typical mix for the high strength concrete required would be approximately 1:2 : 2.5 with a water cement ratio of about 0.40. The maximum sized course aggregate for the girders should be 3/4 inch crushed stone or gravel. The maximum slump should be 5 inches. Such a concrete mix would have an ultimate compressive strength of approximately 5,500 to 6,000 pounds when eight bags of cement per cubic yard are used. With some aggrugates and cements, it should be possible to reduce the cement factor to 6¼ to 7 bags per cubic yard and still obtain the required 5,000 psi.

concrete. High early strength cement should be used to decrease the curing time and the length of time prior to removal of forms. It is the authors' opinion that it would be desirable to use an admixture of 1% "Plastiment" or similar product to make the mix more workable and delay the initial setting time. This admixture will assist in placing and vibrating the concrete in the narrow I section forms.

E. Bridge Deck Concrete

The design of the composite prestressed girder permits employment of deck slab concrete of an ultimate strength normally used for ordinary reinforced concrete construction. The design of the deck slab, diaphrams and guard rail shall be for concrete having an ultimate compressive strength of 3,750 psi. in accordance with the A.S.T.M. tests for 6 x 12 cylinders. The allowable design stresses for the bridge deck concrete are as follows:

f'_c = 3,750 psi.

f_c = .33 f'_c = 1,250 psi.

Extreme fiber tension = 0 psi.

Shear -

Anchored bars, no web steel = .03 f'_c = 112.5 psi.

Anchored bars, with web steel = .06 f'_c = 223.0 psi.

Bearing - Bridge seats = 1,000 psi.

E_c = 1000 f'_c = 3,750,000.

n = 10, assumed value.

The design of the concrete mix and placing of the concrete

by governed by the A.A.S.H.O. Specification, Section 4.

F. High Strength Steel

The prestressed concrete girders are to bd designed for stranded, high strength steel cables similar to those manufactured by the J. A. Roebling Co. and the American Steel And Wire Co. The High strength stranded cables were selected because of the greater speed and ease of construction for the advanced base bridges. To further reduce the construction time the cables are to be unbonded and the prestressing force applied subsequent to pouring of the concrete. Prevention of bond of the steel cables shall be assured by use of plastic or metal tubes and a suitable petroleum lubriant.

The high strength steel cables s hall havw galvanized strands and conform to the following specification.

Ultimate Tensile Strength = 250,000 psi.

Permanent strain less than 0.1% at
70% ultimate tensile strength; or a
mimumum tensile stress of 180,000 psi.
at an elongation of 0.7%.

Initial Allowable stress at the time of prestressing
= 140,000 psi.

Prestress remaining after creep, plastic flow and
shrinkage = $n P_1$ = .85 x 140,000 = 119,000 psi.

The end fittings and bearing plates shall be designed to fully develop the permissible initial stress in the high strength steel stranded cable. The thickness of the steel plate shall be computed by the American Institute of Steel Construction formula:[7]

$$t^2 = .15\ p\ n^2$$

Where; t = thickness of plate in inches

p = bearing pressure in kips per square inch.

n = larger projection of the plate beyond the assumed concentrated load.

G. Reinforcing Steel

The steel for ordinary reinforced concrete design shall be intermediate grade with an allowable tensile stress of 20,000 psi. Ordinary reinforced concrete design theory shall be used in computation of the required stirrups in the web and end block of the precast, prestressed girders.

H. Cracking Factor and Ultimate Factor

The cracking Factor and the Ultimate Factor shall be computed in accordance with Section II, Design Theory, of this thesis and shall have the following minimum values:

Cracking Factor = 1.5 x live Load plus Impact

---

7. Steel Construction Handbook, American Institute of Steel Construction, Fifth Edition, 1951, New York, New York

Ultimate Factor = 2.5 x Live Load Plus Impact

I. Deflections

The total deflection resulting from the cable effect, dead load and live load shall not exceed the ratio 1/800 of the span, the span length being considered the distance center to center of bearings. The dynamic effect of impact loading was not computed in determing the deflections due to the indefinite nature and temporary condition of the impact loading.

J. Advanced Base Requirements

The design of the precast, prestressed girders for advanced base construction imposes additional requirements or criteria. The steel forms shall be prefabricated, designed for quick assembly by parts, and have a small shipping cubage. Typical steel forms are shown in figure 2. The form material for the bridge deck and guard rail shall be that normally available at the advanced base; and, therefore, plywood and timber wales and bracing are indicated in the design.

In order to permit handling and placing of the precast, prestressed girders by construction equipment normally available at the advanced base, the maximum permissible weight of a single girder shall be limited to 50 tons. This allows the placing of the larger girders on the piers or abutments by the use of two 25 ton capacity crawler or floating cranes,

each lifting one end of the girder or one half the load. The smaller girders are to be designed for lifting by a single crane employing a two point pick and spreader beam.

To reduce the number and sizes of stranded high strength cables required at advanced bases, the design shall be made for use of 0.6 inch and 1.0 inch diameter cables.

IV DESIGN OF THE PRESTRESSED GIRDER FOR THE FORTY FOOT SPAN BRIDGE

A. Introduction

The design of the prestressed girder and composite girder for the forty foot span bridge was accomplished in accordance with the design theory formulae of Part II and the specifications and criteria of Part III of this thesis. Numerous trials were required for the proper proportioning of the I section beam and the Tee section composite girder in order to satisfy the many condition equations. The final girder and composite girder are shown in Figure 5 and the centerline stress conditions are shown in Figure 6.

B. Design Calculations

The design calculations for the forty foot span girder and composite girder are set forth in detail in the following paragraphs. To avoid duplication of the theoretical equations, the reference equation or formula number of Part II is shown in parenthesis opposite the particular calculation. The girder section dimensions used in the calculations are shown in Figure 5.

1. Loads:

The maximum live load lane moment for H 20 - S16 - 44 loading equals 449.8 foot kips for the forty foot simple span, and the live load shear equals 55.2 kips.

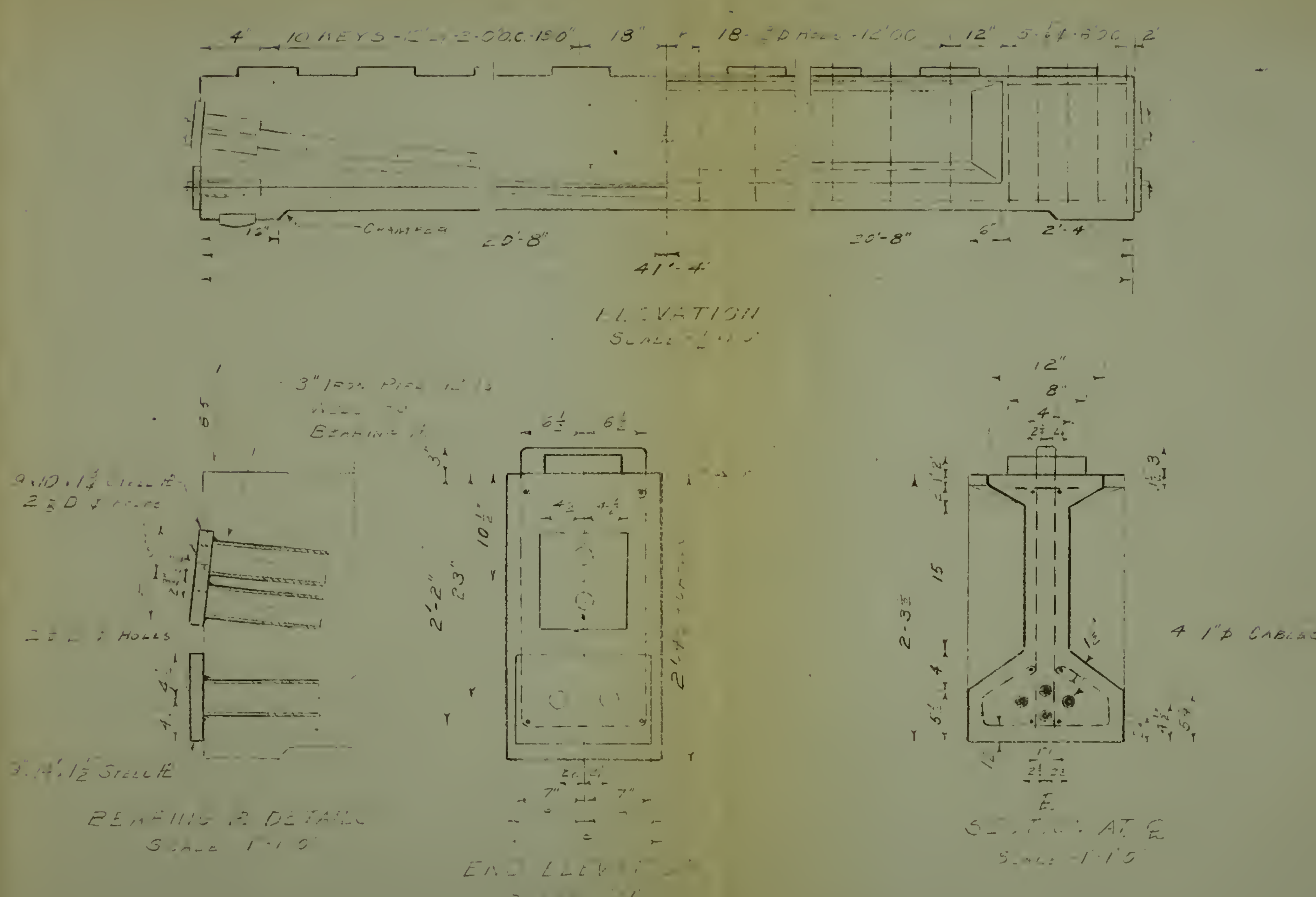

FIGURE No. 5 - DETAILS OF 40' GIRDER

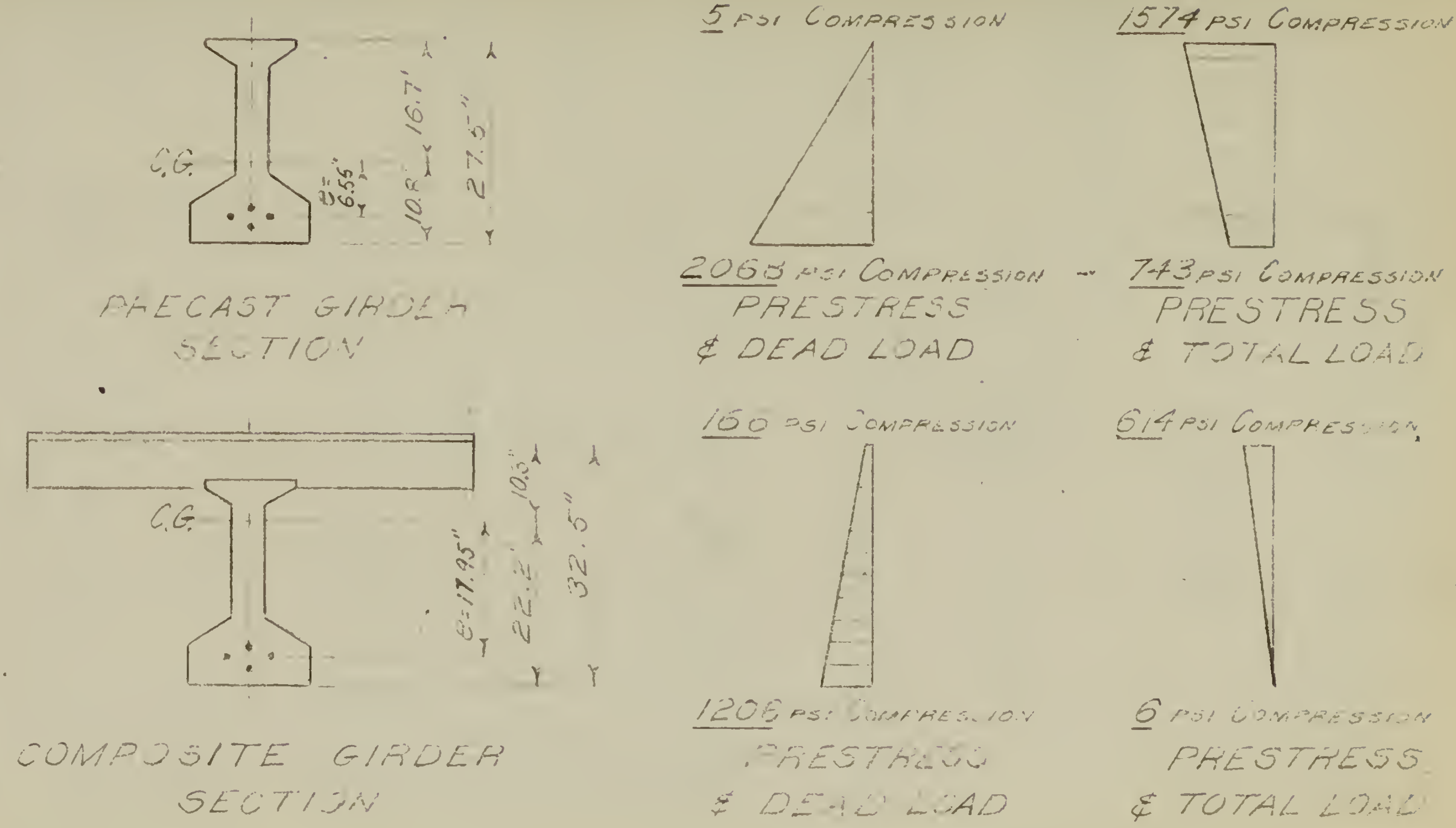

FIGURE NO. 6 STRESS CONDITIONS IN THE 40' GIRDER

$$\text{Impact Factor} = \frac{50}{40 + 125} = 1.3$$

Live load Moment per girder:

$$M_a = \frac{449.8 \times 1.3 \times 12}{2.5} = 2{,}810 \text{ in. kips.}$$

Shear per girder:

$$V_a = \frac{55.2 \times 1.3}{2.5} = 28.7 \text{ kips}$$

2. Section Properties.

Required section modulus:

$$S_b = \frac{2{,}810 \times 1000}{0.775 \times 2{,}000} = 1{,}815 \text{ in.}^3 \qquad (8)$$

Computed Section Modulus:

Composite Section Moment of inertia about the base as reference. (10)

	A	y	Ay	Ay^2	Icg
Base	88.0	2.75	242.0	666.	222.0
Base Triangles	23.	6.83	157.0	1,072.	20.
Web	94.5	16.0	1,512.	24,192.	3,470.
Top triangles	7.5	25.83	194.	5,011.	1.6
Slab	360.	29.5	10,620.	313,290.	1,080.0
total	573.0		12,725.	344,231.	4,794.6

Girder Section

Girder	213.0		2,105	30,941.	3,714
1x 12 plate	12.0	27.0	324	8,748.	1.0
total	225		2,429	39,689.	3,715.

Composite Section Properties

$$I \text{ about base} = 349,025$$

$$Ay^2 = \underline{282,601}$$

$$Icg, \quad 66,424 \text{ in.}^4 \qquad (12)$$

$y_b = 22.2$ in. (11)

$y_t = 10.3$ in.

$A = 572$ sq. in.

$$r^2 \sqrt{\frac{I}{A}} = \frac{66,424}{573} = 159.2$$

$S_t = 6448.9 \text{ in.}^3$

$S_b = 2992.1 \text{ in.}^3$

Girder Properties

$$Y_b = \frac{2429}{225} = 10.8 \text{ in.} \qquad (11)$$

$Y_t = 16.7$ in

$A = 225$

$$I \text{ about base} = 39,689$$

$$Ay^2 = \underline{26,225} \qquad (12)$$

$$Icg = 17,179$$

$$r^2 = \frac{17,179}{225} = 76.4$$

$S_t = 1029 \text{ in}^3$

$S_b = 1591 \text{ in}^3$

3. Design Moments.

Composite Girder:

$M_a = 2,610$ in. K.

$V_a = 20.7$K

$$M_d = 633 \times \frac{150}{44} \times \frac{40^2}{8} \times 12 = 1,582 \text{ in K.}$$

Girder:

$$M_a = 408 \times 1.5 \times \frac{150}{144} \times \frac{40^2}{8} \times 12 = 1{,}530 \text{ in. K.}$$

$$M_d = 225 \times \frac{150}{144} \times \frac{40^2}{8} \times 12 = 563 \text{ in K}$$

4. <u>Flexural</u> <u>Stresses</u>.

Composite Girder:

Top Fiber

$$C_{dt} = \frac{1582}{6449} = 246 \text{ psi.}$$

$$C_{at} = \frac{2810}{6449} = \underline{436 \text{ psi.}}$$

$$\text{total} \quad 682 \text{ psi.}$$

Bottom Fiber

$$C_{db} = \frac{1582}{2992} = 529 \text{ psi}$$

$$C_{ab} = \frac{2810}{2992} = 940$$

$$\text{total} \quad 1469 \text{ psi.}$$

Girder:

Top Fiber

$$C_{dt} = \frac{563}{1029} = 548 \quad \text{psi.}$$

$$C_{at} \quad \frac{1530}{1029} = 1488 \quad \text{psi.}$$

$$\text{Total} \quad 2036 \quad \text{psi.}$$

Bottom Fiber

$$C_{db} = \frac{563}{1591} = 354$$

$$C_{ab} \quad \frac{1530}{1591} = \underline{963}$$

$$\text{Total} = 1317 \text{ psi.}$$

5. Prestress Force and Eccentricity

Cable eccentricity at midspan:

$$C_2 = \frac{246 \times 85}{1469} = 0.142 \qquad (18)$$

$$e = \frac{1.142 \times 159.2}{\left[10.3 - (.142 \times 22.2)\right]} = 25.4 \text{ in.} \qquad (17)$$

Locate the center of gravity of the cables 4.25 inches from the bottom of the beam for proper spacing of the cables.

Composite girder:

$$e = 22.2 - 4.25 = 17.95 \text{ in.}$$

Girder

$$e = 10.80 - 4.25 = 6.55 \text{ in.}$$

Initial prestress force:

$$P_i = \frac{573 \times 1469}{.85\left(1 + \frac{17.95 \times 22.2}{159.2}\right)} = 283{,}000 \text{ pounds} \qquad (19)$$

$$A_s = \frac{283{,}000}{140{,}000} = 2.02 \text{ sq. in.}$$

Use: 4 - 1" diameter cables

area = $4 \times .577 = 2.30$ sq. in.

Force per cable = 70, 705 pounds.

6. Combined Stresses at Midspan:

Composite Girder:

Top Fiber at prestressing:

$$(1) \quad \frac{283{,}000}{573}\left(\frac{17.95 \times 10.3}{159.2} - 1\right) - 246 = 166 \text{ psi compression}$$

Top Fiber with live load acting:

$$(2) \quad -\frac{.85 \times 283{,}000}{573}\left(\frac{17.95 \times 10.3}{159.2} - 1\right) + 682 = 614 \text{ psi compression}$$

Bottom Fiber at prestressing:

$$(3)\quad \frac{283,000}{573}\left(\frac{17.95 \times 22.2}{159.2} + 1\right) - 529 = 1206 \text{ psi compression}$$

Bottom Fiber with live load acting:

$$(4)\quad \frac{-.85 \times 283,000}{573}\left(\frac{17.95 \times 22.2}{159.2} + 1\right) + 1469 = 6 \text{ psi compression}$$

Girder:

Top Fiber at prestressing:

$$(1)\quad \frac{283,000}{225}\left(\frac{6.55 \times 16.7}{76.4} - 1\right) - 548 = 5 \text{ psi compression}$$

Top Fiber with added load acting:

$$(2)\quad \frac{-.85 \times 283,000}{225}\left(\frac{6.55 \times 16.7}{76.4} - 1\right) + 2036 = 1574 \text{ psi compression}$$

Bottom fiber at prestressing:

$$(3)\quad \frac{283,000}{225}\left(\frac{6.55 \times 10.8}{76.4} + 1\right) - 354 = 2068 \text{ psi compression}$$

Bottom fiber with added loads acting:

$$(4)\quad \frac{-.85 \times 283,000}{225}\left(\frac{6.55 \times 10.8}{76.4} + 1\right) + 1317 = 743 \text{ psi compression}$$

Conditions (1) and (3) above for the composite girder never actually exist because of the precast method of construction. The theoretical stresses have been computed, however, to serve as a basis of comparison with other designs.

7. <u>Bending up the Cables at the End of the Girder.</u>

In order to provide a balanced end block and girder section axial loading condition due to the prestressed cables, the various cables have been bent up as shown in Figure 5.

These parabolic cable trajectories provide adequate cable effect to reduce the end shear to an allowable amount as shown in later calculations. The cable forces are balanced about the center of gravity of the girder section.

$$\text{Force per cable} = \frac{283,000}{4} = 70,750 \text{ pounds.}$$

8. <u>Bearing</u> <u>Plates.</u>

Bottom Bearing Plate:

To permit proper spacing of the cable end fittings and the jacking equipment for applying the prestress force, use a 9 inch by 14 inch plate as shown in Figure 5.

Allowable Bearing = $5\ f'_c$ = 2,500 psi.

Area of Plate:

$$A = 9 \times 14 - 2\pi \frac{3^2}{4} = 112 \text{ sq. in.}$$

Bearing Pressure:

$$P = \frac{2 \times 80,600}{112} = 1,440 \text{ psi.}$$

Thickness of plate:

$$t = \sqrt{.15 \times 1.44 \times 3.0^2} = 1.4''$$

Therefore, use a 9" x 14" plate with a thickness of $1\frac{1}{2}$ inches.

Top bearing plate:

A bearing plate 9 x 10 inches as shown in Figure 5 was chosen to allow proper spacing of cables and end fittings.

Area of Plate

$$A = 9 \times 10 - 2\pi \times \frac{3^2}{4} = 76 \text{ sq. in.}$$

Bearing Pressure:

$$P = \frac{2 \times 80,600}{76} = 2,120 \text{ psi.}$$

Thickness of Plate:

$$t = \sqrt{.15 \times 2.12 \times 3.0^2} = 1.7 \text{ in.}$$

Therefore, use a 9 inch by 10 inch plate with a thickness of 1 3/4 inches.

9. <u>End Shear and The Principal Tensile Stress</u>

Shear at the End of the Span.

Composite Girder:

Live load shear = 28.7 Kips.

Dead Load Shear:

$$V_D = 633 \times \frac{150}{144} \times 20 = 13,200 \text{ pounds}$$

Total Shear = 28.7 + 13.2 = 41.9 Kips.

Girder:

Added Load Shear:

$$V_a = 408. \times 1.5 \times \frac{150}{144} \times 20 = 12,750 \text{ pounds}$$

Dead Load Shear:

$$V_D = 225 \times \frac{150}{144} \times 20 = 4,690 \text{ pounds.}$$

Weight of Girder = 4.69 Tons

Total Shear = 12.75 + 4.69 = 17.44 Kips

Cable Effect:

Cable trajectory is parabolic or $x^2 = 2py$

Ordinate For Top Calbe:

$y = 13.98$ in.

Ordinate for the Middle Cable:

$y = 12.23$ in.

Ordinate for the two bottom cables:

$y = 0$

Top Cable Effect:

$$P = \frac{x^2}{2y} = \frac{240^2}{2 \times 13.98} = 2,060$$

$$\tan \alpha = \frac{240}{2060} = .1163$$

$$\sin \alpha = .1158$$

$$V' = n\ P_1 \sin \ = .85 \times 70,750 \times 1158 = 6,950 \text{ pounds}$$

Middle Cable Effect:

$$P = \frac{240^2}{2 \times 12.23} = 2,350$$

$$\tan \alpha = \frac{240}{2350} = .1022$$

$$\sin \alpha = .102$$

$$V' = .85 \times 70,750 \times .102 = 6,130 \text{ pounds.}$$

Bottom Cables Effect

$$V' = 0$$

Total cable effect or negative shear:

$$V' = 6,950 + 6,130 = 13,080 \text{ pounds.}$$

Net Shear at End of Beam

Composite Girder

(24) $V = 41,900 - 13,080 = 28,820$ pounds.

Girder

(24) $V = 17,440 - 13,080 = 4,360$ pounds.

Resisting Section Above Beam Center of Gravity.

Composite Girder:

$$Q = \sum A_y$$

Slab = 360 x 7.3 = 2630

$$\text{triangles} = 7.5 \times 3.64 = 27$$

$$\text{Web} = \frac{4.3^2}{2} \times 4.5 \qquad \underline{42}$$

$$\text{total} = 2699 \text{ in.}^3$$

Resisting Section for Shear

Girder:

$$\text{Plate} = 12 \times 16.2 = 195$$

$$\text{Tringles} = 7.5 \times 15.04 = 113$$

$$\text{Web.} = \frac{(15.7)^2}{2} \times 4.5 \quad \underline{554}$$

$$\text{total} \quad Q = 862 \text{ in.}^3$$

Unit Shearing Stress:

Composite Girder:

$$(20) \qquad v = \frac{28,820 \times 2699}{66,424 \times 4.5} = 260 \text{ psi.}$$

Girder:

$$(20) \qquad v = \frac{4,360 \times 862}{17,179 \times 4.5} = 48.6 \text{ psi,}$$

Axial Compressive Stress:

Composite Girder:

$$C_x = \frac{N\ P_i}{A} = \frac{.85 \times 283,000}{573} = 419 \text{ psi.}$$

Girder:

$$C_x = \frac{.85 \times 283,000}{225} = 1068 \text{ psi.}$$

Principal Tensile Stress

Composite Girder

$$(25) \qquad pt = \sqrt{260^2 + \left(\frac{420}{2}\right)^2} - \frac{420}{2} = 124 \text{ psi.}$$

Girder:

(25) $$pt = \sqrt{\left(\frac{1068}{2}\right)^2 + 48.6^2} - \frac{1068}{2} = 3 \text{ psi.}$$

No stirrups are required for shear or principal tensile stress, but use 3/8 inch diameter stirrups at 12 inches center, for temperature steel.

10. <u>Shear at the Junction of the Deck slab and the Girder.</u>

$V = 28,820$ pounds, from 9.

$b = 12$ inches

$Q = 360 \times 7.3 - 12 \times 438 = 2050 \text{ in.}^3$

$$v = \frac{2050 \times 28,820}{66,424 \times 12} = 74.2 \text{ psi.}$$

Use shear key blocks two inches in height and 12 inches long as shown in Figure 5. to assure development of sufficient shearing strength at the junction of the girder and the slab. The actual shearing stress then equals:

$$v = 74.2 \times 2 = 148.4 \text{ psi.}$$

To further assure contiguous or integral action between the slab and the girder extend the 3/8 inch diameter girder stirrups into the floor slab as shown in Figure 5. Proper tie is assured at the end block by extending the 7/8 inch diameter stirrups into the slab.

11. <u>Stresses at the end of the Girder;</u>

Bending moment = 0

Girder:

The cables provide a balanced loading about the center of gravity of the girder and, therefore, the compressive stress due to the prestress force is uniform throughout the section.

$$c_x = \frac{.85 \times 283{,}000}{225} = 1068 \text{ psi. compression}$$

Composite Girder:

Top Fiber Stress

(2) $$\frac{.85 \times 283{,}000}{573} - \frac{.85 \times 283{,}000 \times 11.4 \times 10.3}{66{,}424} = 7 \text{ psi. tension}$$

Bottom Fiber Stress

(4) $$\frac{.85 \times 283{,}000}{573} + \frac{283{,}000 \times 11.4 \times 22.2 \times .85}{66{,}424} = 1{,}336 \text{ psi. compression}$$

12. The cracking Factor.

Modulus of Rupture of Concrete = 600 psi.

(26) $$\frac{-.85 \times 283{,}000}{573} \left(\frac{17.95 \times 22.2}{159.2} - 1 \right) + CF \times 940 + 529 = 600$$

$$CF = \frac{1541}{940} = 1.64$$

The first crack is calculated to occur in the bottom fiber when the dead load is acting plus the additional loading of 1.64 times the live load plus impact.

13. Ultimate Factor.

Prestress force for four cables at 80% of the guaranteed ultimate strength:

$P = .8 \times 4 \times 122{,}000 = 391{,}000$ lbs.

$e = 17.95$ in.

$y_t = 10.3$ in.

Area for failure at top fiber:

$$A = \frac{391{,}000}{5{,}000} = 78.20$$

Depth of Area.

$$d_1 = \frac{78.20}{60} = 1.30''$$

Total Effective depth;

$$jd = 17.95 + 10.3 - \frac{1.30}{2} = 27.6''$$

Resisting Moment:

$$M = 391,000 \times 27.6 = 10,800,000 \text{ in. lbs.}$$

Ultimate Factor

$$10,800 = UF\ (2810) + 1,582$$

$$UF = 3.31$$

Total failure of the structure occurs under the loading of dead load plus 3.31 times the live load plus impact.

14. End Block Stresses.

The dimensions and cable arrangement at the end block are shown in Figure 5. In accordance with the theory presented in Part II, the following loads are considered to be acting for maximum stress conditions.

$$P_i = 283,000 \text{ lbs.}$$

$$P \text{ per cable} = 70,750 \text{ lbs.}$$

Bearing Plate Loads:

Top Plate:

$$P_1 = \frac{2 \times 70,750}{10} = 14,150 \text{ lbs./in.}$$

Bottom plate:

$$P_2 = 2 \times 70,750 = 15,720 \text{ lbs./in.}$$

Girder Loads:

Normal Stress

$$C_x \quad \frac{283{,}000}{225} = 1{,}258 \text{ psi.}$$

Top Plate = 15, 100 lbs/in.

Web = 5, 660 lbs/in.

Bottom Plate = 20,128 lbs/in.

$$\text{Top Transition} = \frac{15{,}100 + 5660}{2} = 10{,}380 \text{ lbs/in.}$$

$$\text{Bottom Transition} = \frac{20{,}128 + 5660}{2} = 12{,}894 \text{ lbs./in.}$$

The end block loads, shear, and moment are shown in Figure 15. The maximum shear occurs at Condition I and the maximum moment occurs at Condition II. These maximum sections are used for computation of the principal tensile stress.

End Block Axial Stress;

$$C_x = \frac{P_i}{A} = \frac{283{,}000}{16 \times 27.5} = 643 \text{ psi compression}$$

Shear - Resulting from dead and live loads:

$$v = \frac{3}{2} \frac{V}{bh} = \frac{3}{2} \times \frac{28{,}820}{16 \times 27.5} = 98.3 \text{ psi.}$$

Normal Stress Due to Prestress Force.

Condition I:

(28) $$C_z = \frac{M}{ba^2} K = \frac{148{,}691}{16 \times (275)^2} = 12.3 \quad K$$

Condition II:

(28) $$C_z = \frac{284{,}544}{16 \times (275)^2} = 28.95 \text{ K}$$

Shear Due to Prestress Force:

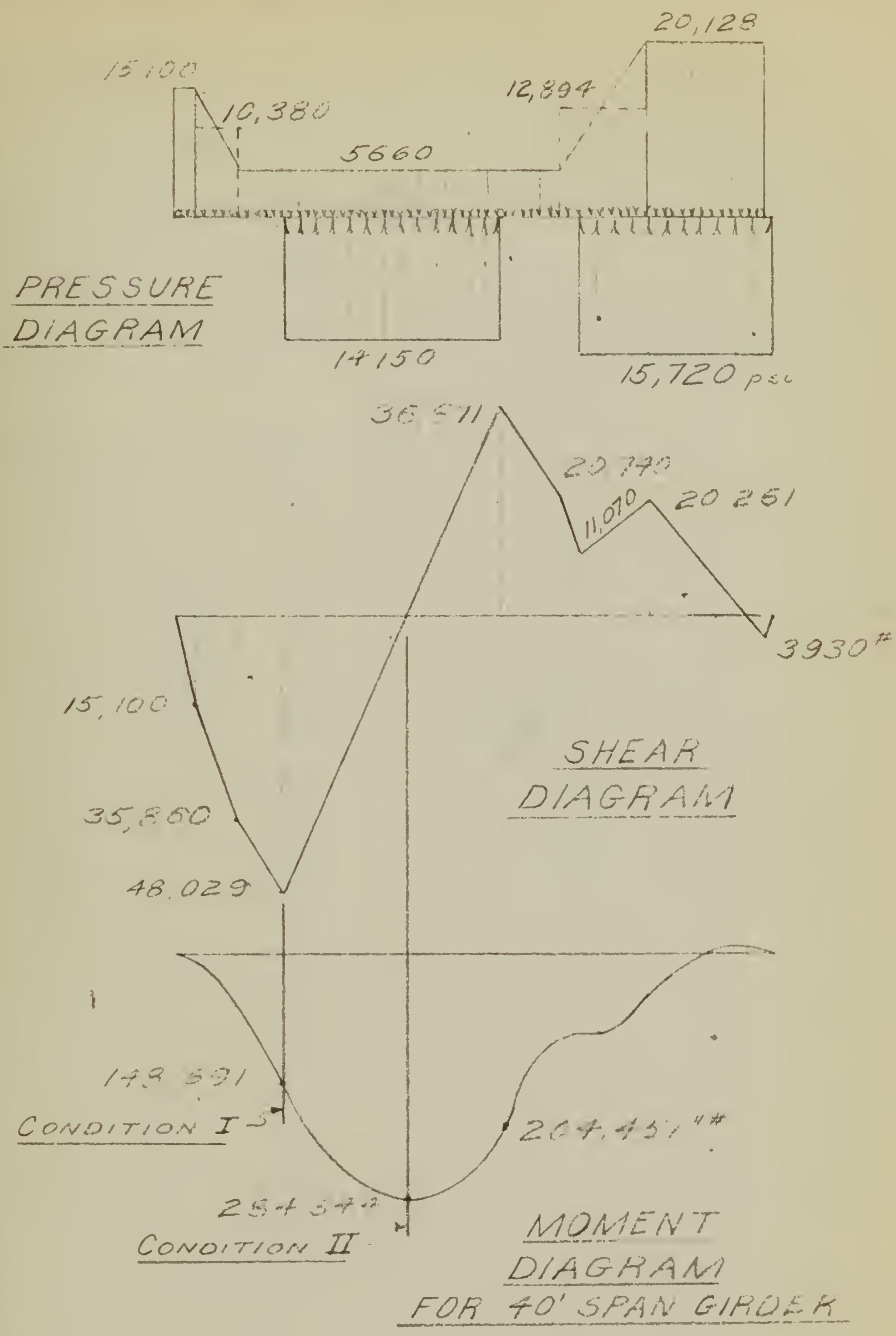

FIGURE No. 15 END BLOCK FORCES

Condition I:

(29) $v = \frac{V}{ba} \quad K_1 = \frac{48,029}{16 \times 27.5} = 109.1 \; K_1$

Condition II:

(29) $v = 0 \times K_1$

Principal Tensile Stress:

Typical Calculation:

(30) $P_t = \frac{643.0 - 61.5}{2} - \sqrt{(234.6)^2 + \left(\frac{643.0 + 61.5}{2}\right)^2}$

$= 133$ psi tension

Tabulation of Principal Tensile Stress:

Condition I:

3/a	C_x	C_z	v	Pt
-0.5	643.0	0.0	0 + 98.3 = 98.3	
-0.4		-6.4	2.0 + 98.3 = 100.3	
-0.3		-21.6	14.0 + 98.3 = 112.3	
-0.2		-39.8	41.3 + 98.3 = 139.6	
-0.1		-55.2	83.8 + 98.3 = 182.1	
0.0		-61.5	136.3 + 98.3 = 234.6	133
+0.1		-53.1	188.5 + 98.3 = 286.8	156 *
+0.2		-24.1	224.2 + 98.3 = 322.5	155
+0.3		+31.5	230.0 + 98.3 = 328.3	111
+0.4		+119.8	223.5 + 98.3 = 321.8	
+0.5		+246.0	159.0 + 98.3 = 257.3	

* Maximum principal tensile stress equals 156 psi for Condition I.

Condit on II

z/a	C_x	C_z	v	pt
-0.5	643.0	0	0 + 98.3 = 98.3	
-0.4		-15.1		
-0.3		-50.9		
-0.2		-93.6		
-0.1		-129.5		
0.0		-144.5		160
+0.1		-125.0		
+0.2		-56.7		
+0.3		+74.0		
+0.4		+281.5		
+0.5		+579.0		

$$(30)\quad pt = \frac{643.0 - 144.5}{2} - \sqrt{98.3^2 + \left(\frac{643. + 144.5}{2}\right)^2} = 160 \text{ psi tension}$$

The absolute maximum principal tensile stress equals 160 psi.

Assume the principal tensile stress is a maximum at an angle of 45° with the horizontal axis and provide sufficient reinforcing steel to resist the total principal tensile stress. This ddsign will assure that no tensile cracks occur in the concrete end blocks.

Area of steel required:

$$A_s = \frac{12 \times 16 \times 160}{.707 \times 20{,}000} = 2.18 \text{ sq. in per foot.}$$

Use two 7/8 inch diameter stirrups per foot or on 6

inch centers. Resulting steel area equals 2.40 sq. in. per foot of length. Extend the stirrups into the floor slab as shown in Figure 5. to provide adequate resistance to shear.

For lifting the beam utilize the center 7/8 inch diameter stirrup of the group at each end of the girder.

Strength of Stirrup = .6 x 20,000 = 12,000 lbs.

Weight of one half girder $= \frac{4.69 \times 2{,}000}{2} = 4{,}690$ lbs.

15. Girder and Composite Girder Defections

Girder Defections at the Midspan:

Use E_c = 3,000,000 and assume gross section effective.

Cable Effect:

$$(37)\ \Delta = \frac{.65 \times 283{,}900 \times 6.55 \times 40^2 \times 144.5}{3{,}000{,}000} = \pm .737 \text{ in.}$$

Dead Load, Slab Load and Construction Load:

Weight per foot $= (1.5 \times 408 + 225) \times \frac{150}{144} = 872$ lbs/ft

$$(31)\ \Delta = \frac{5 \times 872 \times 40 \times 40^3 \quad 1728}{384 \times 3{,}000{,}000 \times 17{,}179} = -.975$$

Total Deflection:

$$\Delta = -.975 + .737 = -.238 \text{ in.}$$

Ratio of Deflection to Span length:

$$\frac{\Delta}{L} = \pm\frac{1}{2020}$$

Net deflection upon removal of the forms and the construction load:

$$\Delta = +.738 - \left(\frac{660}{837}\right).975 = -.021 \text{ inches}$$

Composite Girder Deflections at the Midspan:

Theoretical Cable Effect:

This condition does not exist for the precast construction method.

$$(37)\ \Delta = \frac{.85 \times 283{,}000 \times 17.95 \times 40^2 \times 144 \times 5}{3{,}000{,}000 \times 66{,}424 \times 48} = +.520 \text{ in.}$$

Theoretical Dead Load Deflection:

This condition does not exist for the precast method of construction.

$$(31)\ \Delta = \frac{5 \times 633 \times 150 \times 40 \times 40^3 \times 1728}{384 \times 144 \times 66{,}424} = -\ 0.191 \text{ in.}$$

Live Load Deflection:

The Truck loading will govern so place the H20 - S16-44 truck-trailer on the span in such a manner that the rear wheel of the tractor is at the midspan point for maximum moment and deflection. The resulting axel loads per beam are:

$$\text{Front Axel} = \frac{8.0}{2.5} = 3.2 \text{ Kips}$$

$$\text{Middle Axe.} = \frac{32.0}{2.5} = 12.8 \text{ Kips}$$

$$\text{Rear Axel} = \frac{32.0}{2.5} = 12.8 \text{ Kips.}$$

The resulting end reactions of the composite girder are:

Left end = 11.04

Right end = 17.76

$$(32)\quad \Delta = \int_0^{l} \frac{M_m\,dx}{EI}$$

$$\Delta \frac{x\ 3{,}000{,}000 \times 66{,}424}{1000} = \int_0^6 5.52\,x^2\,dx + \int_0^{14} 198.72\,dx + 56.64x\,dx$$

$$+ 392\,x^2\,dx + \int_0^6 8.88\,x^2\,dx + \int_0^{14} 320\,dx + 68.13\,x\,dx$$

$$+ 2.48\,x^2\,dx$$

$$\Delta = -\ .229 \text{ in.}$$

Total Deflection or Maximum Deflection

The total deflection is equal to the cable effect for the girder, plus the net deflection of the girder due to dead loads of the structure, plus the live load deflection of the composite girder.

$$\Delta t = t.\ 737 - .758 - .229 = -\ .250 \text{ in.}$$

Ratio of Defledtion to Span Length

$$\frac{\Delta}{L} = \frac{.250}{40 \times 12} = \frac{1}{1920}$$

16. Ratio of Depth to Span Length

Girder:

$$\frac{D}{L} = \frac{27.5}{40 \times 12} = \frac{1}{17.5}$$

Composite Girder

$$\frac{D}{L}\ \frac{32.5}{40 \times 12} = \frac{1}{14.7}$$

C. Conclusions

The design calculations indicate that the girder and composite girder for the forty foot span bridge exhibit all of the desirable properties typical of prestressed concrete construction. The girders have small deflections under the design loads, are relatively shallow in depth, and are light in weight for concrete members. The thin web members permits the most efficient use of the high strength concrete, as shown by Figure 6. The arrangement of the prestressed cables is relatively simple and easy to install.

Interesting features of the forty foot span bridge are the shapes or proportions of the sections of the girder and the composite girder. For the girder, the added load moment is approximately three times as great as the dead load moment of the girder itself. Efficient use of the high strength concrete results in an inverted Tee section as shown in Figure 5. Utilizing a section of the floor slab as part of the composite girder the resulting section has Tee proportions. This Tee section effectively resists the prestress force, live load moment plus impact and the dead load moment. The compressive stress at the top fiber of the composite girder is considerably less than the 1,250 psi. Allowable. The principle tensile stress is well within the allowable specification of 150 psi for the composite girder and is almost negligible for the simple girder.

In order to obtain the proportions of the sections shown

in Figure 5 numerous trials were required. The seventeen independent condition equations shown in the design calculations for each girder and each composite girder tax the designer's ingenuity and resourcefulness. Prior experience in the design of prestressed linear members, steel design, and ordinary reinforced concrete design greatly assist the designer in judging the proper approach for modification of preliminary trail sections. The choosing of a suitable section for the forty foot span girder was particularly difficult due to the rather wide range of dead and live load moments acting on the structure.

The saftey factors against cracking and total failure of the composite girder are considered adequate for advanced base temporary construction and also for permanent continental construction projects. Cracking of the bottom fiber concrete is calculated to occur when the dead load and 1.64 times the live load plus impact are applied to the structures. Total failure to destruction is calculated to occur under the dead load of the structure and 3.31 times the live load plus impact.

V TABULATION OF DESIGN CALCULATIONS FOR THE SIXTY, EIGHTY, ONE HUNDRED AND ONE HUNDRED TWENTY FOOT SPAN BRIDGES

A. Introduction

The design of the forty, sixty, eighty, one hundred and one hundred twenty foot span bridges was accomplished in a manner identical with the forty foot span bridge explained in detail in part IV above. A tabulation of the results of the various steps in the design are given below and a comparison with corresponding steps numbers in part IV will furnish the necessary formulus and explanation. Details of the final girder designs are shown in Figures 7 through 14.

B. Design Calculation Results

	60' Span	80' Span	100' Span	120' Span
1.	Loading, Moment and Shear			
M	806.5 ft K.	1164.9 ft.K	1524 ft K	1883 ft K
V	60.8 K	63.6	65.3 K	66.4 K
I.L.	27 %	24.4 %	22%	20.4%
M_a	4920 in K	6950 in K	8940 in K	10880 in K
V_a	30.9 K	31.7 K	31.9 K	32 K
2.	Section Properties Composite Section			
I_{cg}	142785 in^4	208393 in^4	391.080 in^4	617587 in^4
y_t	14.2 in	16.3 ih	22 in	25 in
y_b	27.8 in	31.7 in	37 in	42 in

K stands for Kips

Design Calculation results (continued)

	60' Span	80' Span	100' Span	120' Span
2. Section Properties				
Composite Section				
A	670 sq. in	780 sq in	928 sq in	1184 sq in
r^2	213 sq in	267 sq in	421 sq in	522sq in
Girder				
I_{cg}	52067 in^4	96863 in^4	193900 in^4	393,867 in^4
yt	21.05 "	21.1 in	27.93 in	28.47 in
yb	15.95 "	21.9 in	26.07 in	33.53 in
A	328 sq in	438 sq. in	587 sq in	856 sq in
r^2	158.5 sq in	220.5 sq in	331 sq in	460 sq in
3. Design Moments				
Composite Section				
M_a	4920 in K.	6950 in K	8940 in K	10880 in K.
M_d	4262 in K.	8680 in K	16000 in K	29995 in K.
Girder				
M_a	3632 in K.	6310 in K	9960 in K	15055 in K
M_d	1850 in K.	4380 in K	9.75 in K	19250 in K
4. Flexural Stresses at the Center of the Span				
Composite Section				
C_{dt}	490 psi	683 psi	902 psi	1212 psi
C_{at}	425 psi	547 psi	503 psi	440 psi
Sum	815 psi	1230 psi	1405 psi	1652 psi
C_{db}	960 psi	1318 psi	1517 psi	2040 psi
C_{ab}	833 psi	1055 psi	847 psi	738 psi
Sum	1798 psi	2373 psi	2364 psi	2778 psi

Design Calculation results (continued)

	60' Span	80' Span	100' Span	120' Span
	Girder			
C_{dt}	1470 psi	992 psi	1320 psi	1392 psi
C_{at}	749 psi	1430 psi	1432 psi	1088 psi
Sum	2219 psi	2422 psi	2752 psi	2480 psi
C_{db}	1120 psi	958 psi	1233 psi	1637 psi
C_{ab}	567 psi	1375 psi	1340 psi	1280 psi
Sum	1687 psi	2331 psi	2573 psi	2917 psi
5.	Prestress Force and Eccentricity			
	Composite Section			
	23.3 in	25.36 in	32.0 in	36 in
	Girder			
	11.47 in	14.87 in	21.0 in	27.5 in
	Both			
P_i	350,000 lb	544,000 lb	677,000 lb	992,000 lb
Use	4 - 1" Ø 1 - 0.6in Ø	7 - 1 in Ø	8 - 1-in Ø 2 - 0.6 in Ø	12 - 1 in Ø
6.	Combined Stresses at the Center of the Span			
	Composite Section			
(1)	135 psi	292 psi	414 psi	605
(2)	569 psi	898 psi	990 psi	1137
(3)	1282 psi	1477 psi	1258 psi	1225
(4)	6 psi	12 psi	4 psi	0
	Girder			
(1)	192 psi	396 psi	432 psi	575 psi
(2)	1748 psi	1916 psi	1997 psi	1795 psi
(3)	1728 psi	2054 psi	1827 psi	1838 psi
(4)	272 psi	229 psi	32 psi	38 psi

Design Calculation results (continued)

	60' Span	80' Span	100' Span	120' Span
7.	Bending up of cables at the end (Cable effect)			
	Both			
P	2-1in-80600 lb	5 - 1 in66200 lb	4-1 in-77,200	8,-1-in 82,600 lb
[illegible]	1-.6 in-27600 lb		2-.6in-29,700	
V'	18825 lb.	28460 lb.	38820 lb	63,300 lb.
8	Bearing Plates			
th	1.46 in	1.69 in	1.78 in	1.78 in
use	9x6 x $1\frac{1}{2}$ in	10 x 9 x $1\frac{1}{2}$ in	10 x $16\frac{1}{2}$ x 13/4 in	10x$16\frac{1}{2}$x13/4in
th	0.885 in	1.46 in	0.867 in	1.78 in
use	9 x 6 x 1 in.	9 x 6 x $1\frac{1}{2}$ in	9 x $13\frac{1}{2}$ x 7/8 in	10x$16\frac{1}{2}$x13/4 in
th	1.76 in	1.76 in	1.78 in	1.78 in
use	6 x 14 x 13/4 in	6x14x13/4 in	10x$16\frac{1}{2}$x 13/4in	10 x 19x13/4in
9	End Shear and Principal tensile stress			
	composite Section			
VL	30,900 lb	31,700 lb	31900	32000 lb
VD	22,830 lb	35900	51500	77800 lb
D	422 lb	900 lb	1350	4500 lb
V'	-18825 lb	-28460 lb	-38820	-63300 lb
Net V	35327 lb	39140 lb	45930	51000 lb
Q	4377 lb	5678 in^3	8691 in^3	12218 in^3
P_t	217 psi	178 psi	146 psi	126 psi
P_t	89 psi	49 psi	33 psi	22 psi
	Girder			
VL	18800 lb	25100 lb	31300 lb	36400 lb

Desigh Calculation results (continued)

	60' Span	80' Span	100' Span	120' Span
VD	10260 lb	18300 lb	30550 lb	53500 lb
D	422 lb	900 lb	1350 lb	4500 lb
V'	-18825 lb	-28460 lb	-38820 lb	-63300 lb
Net V	10657 lb	15840 lb	24380 lb	31100 lb
Q	1865 in^3	2969 in^3	4823 in^3	8380 in^3
v	76 psi	81 psi	87 psi	83 psi
P_t	5 psi	6 psi	7 psi	7 psi
10	Shear at the junction of the deck slab and the girder.			
Q	3874 in^3	4634 in^3	6543 in^3	7306 in^3
v	53 psi	48 psi	43 psi	19 psi
use per feet	2 x 8 x 12 in shear blocks 2-3/8Ø a)12in OC	2 x 8 x 12 in shear blocks 2-3/8Ø/a) 12in OC	2 x 8 x 12in shear blocsk 2-3/8Ø a) 12 in OC	2 x 8 x 12 in shear blocks 2-3/8Ø a) 12 in OC
12	Stresses at the end of the beam			
	Girder			
C_x	907 psi	1053 psi	980 psi	980 psi
	Composite Section			
f_t	94 psi	594 psi	266 psi	423 psi
f_b	1130 psi	977 psi	1216 psi	1194 psi
12	The cracking Factor			
CF	1.63	1.58	1.71	1.81
13	Ultimate Factor			
	2.34	2.75	3.27	3.60

Design Calculation result (continued)

	60' span	80' Span	100' Span	120' Span
14	End Blcok Stresses			
I-M	245670 in lb	558,270 in lb	1,160,513 inlb	1,113,899 in lb
I-V	0	0	0	0
II-M	152154 in lb	95,231 in lb	557,888 in lb	421,784 in lb
II-V	50,718 in lb	61,842 lb	126,784 lb	164,990 lb.
C_x	512 psi	702 psi	695 psi	800 psi
v	77.4 psi	75.8 psi	70.8 psi	61.7 psi
$I-_{cz}$	5.15 psi tension	91.5 psi	111.0 psi	27.4 psi
II_{cz}	27.6 psi	5.6 psi	20.7 psi	62.5 psi
I-v	0	0	0	0
II-v	140 psi	164.5 psi	268 psi	230.0 psi
P_t	II-115 psi	I - 98 psi	II - 152 psi	II - 151 psi
Per ft As	2-~~7/8∅~~ 3/4" -6in OC	2-3/4in∅ -6in OC	2-7/8in∅-6inOC	2-7/8in∅-6in OC
WT	10,260 lb	18,500 lb	31,000 lb	54,000lb
Lift	center stirrup eq. end block	1 in∅ U-Bolt each end	1½ in U-Bolt each end	1 7/8in U Bolt each end
15	Deflections at Midspan Girder			
CE	-1.18 in	-2.29 in	-3.11 in	-4.23 in
max Load	1.81 in	3.43 in	4.82 in	5.95 in
Net Load	1.42 in	2.79 in	3.96 in	5.15
≤ Δ	0.63 in	1.14 in	1.71 in	1.72 in

Design calculations results (continued)

	60' Span	80' Span	100 ' Span	120' Span
	Composite			
A_{LL}	0.42 in	0.69 in	0.78 in	0.87 in
$\Sigma\Delta$	0.66 in	1.19 in	1.63 in	1.77 in
$\frac{\Delta}{L}$	1:1090	1:800	1:737	1:815
16	Ratio of Depth to Span Length			
	Composite Section			
D/L	1:17.2	1:20	1:20.4	1:21.5
	Girder			
D/L	1:19.5	1:22.3	1:22.2	1:23.2

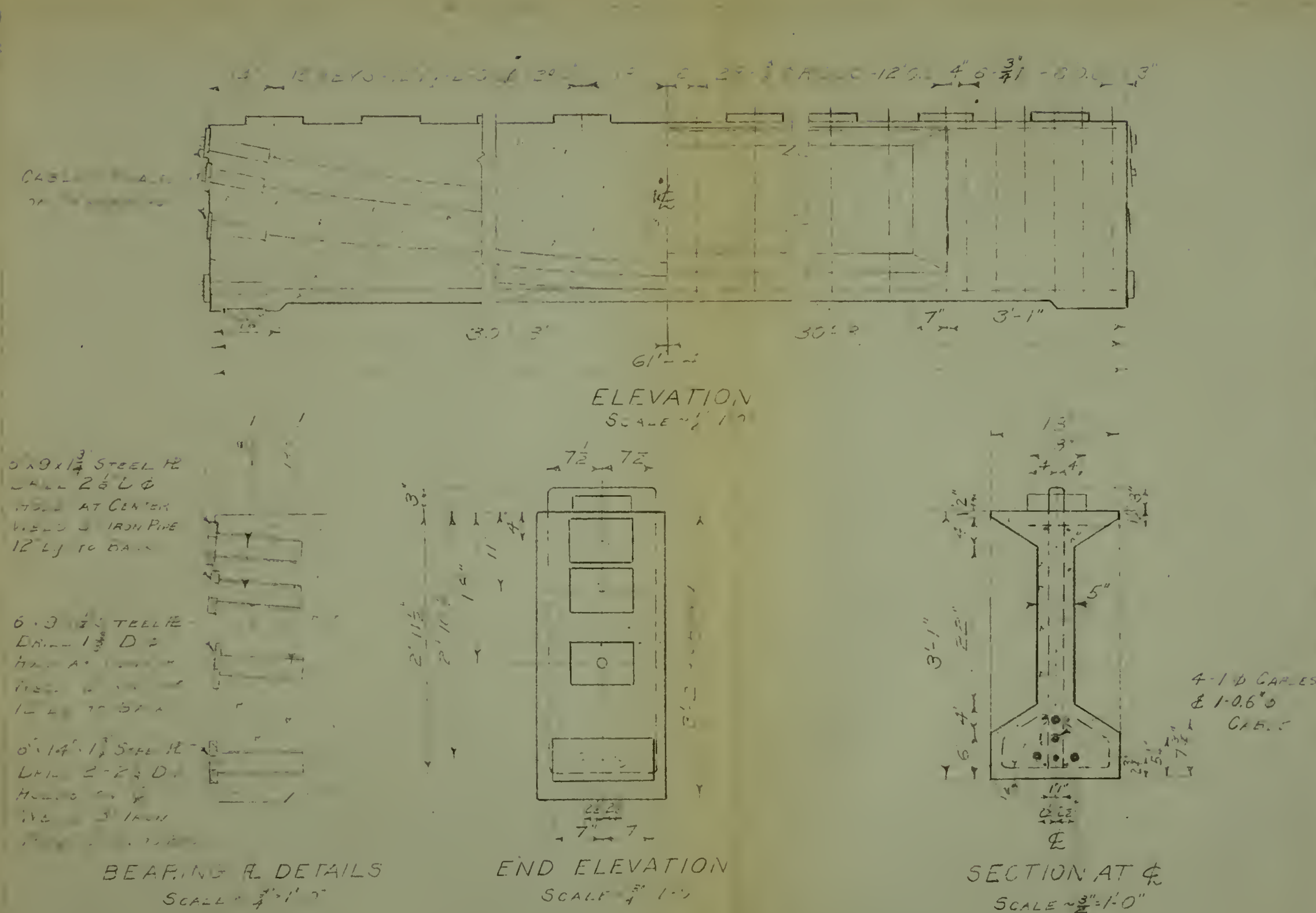

FIGURE No. 7 DETAILS OF 60' GIRDER

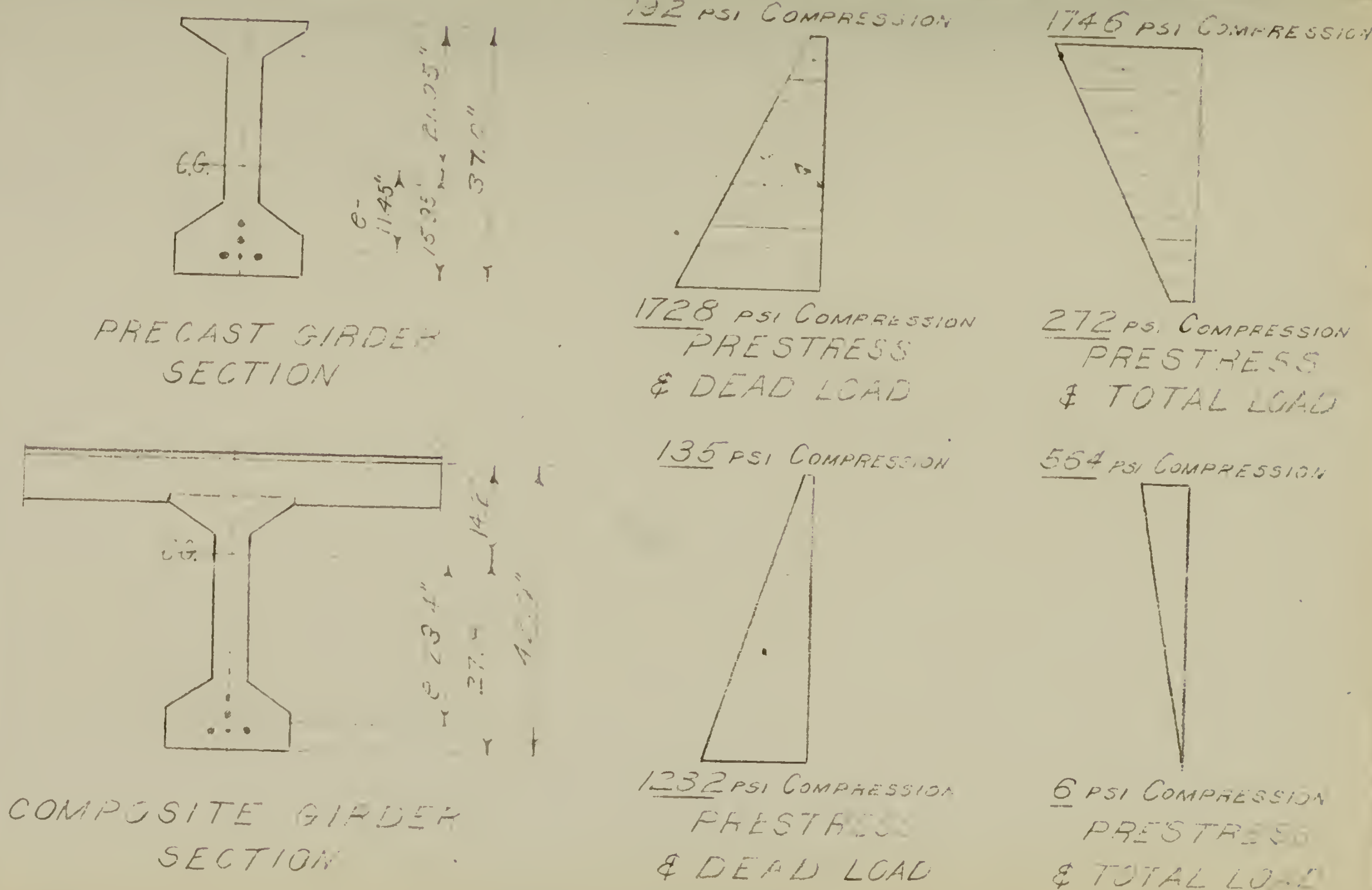

FIGURE NO. 8 STRESS CONDITIONS IN THE 60' GIRDER

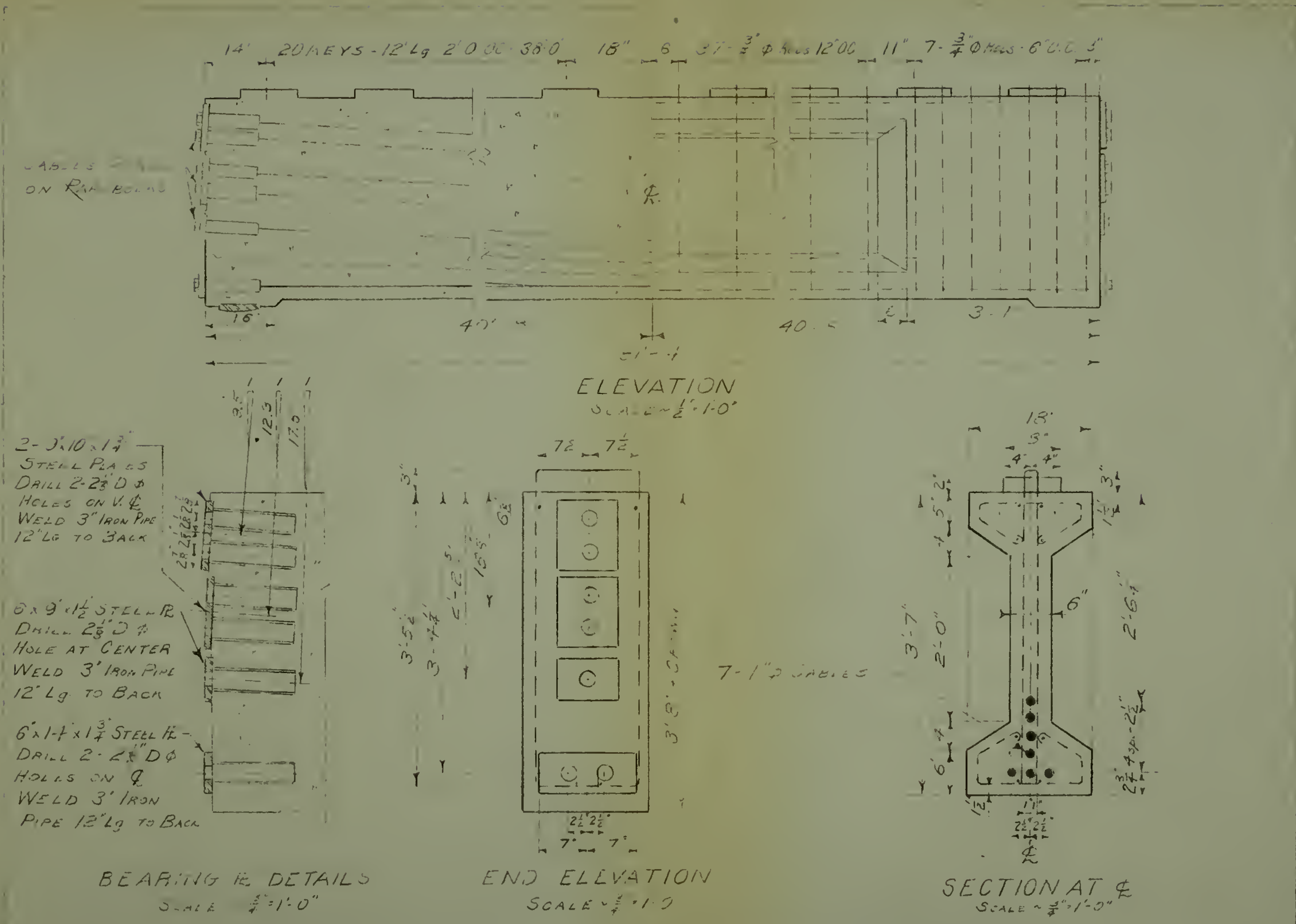

FIGURE No 9 DETAILS OF 80' GIRDER

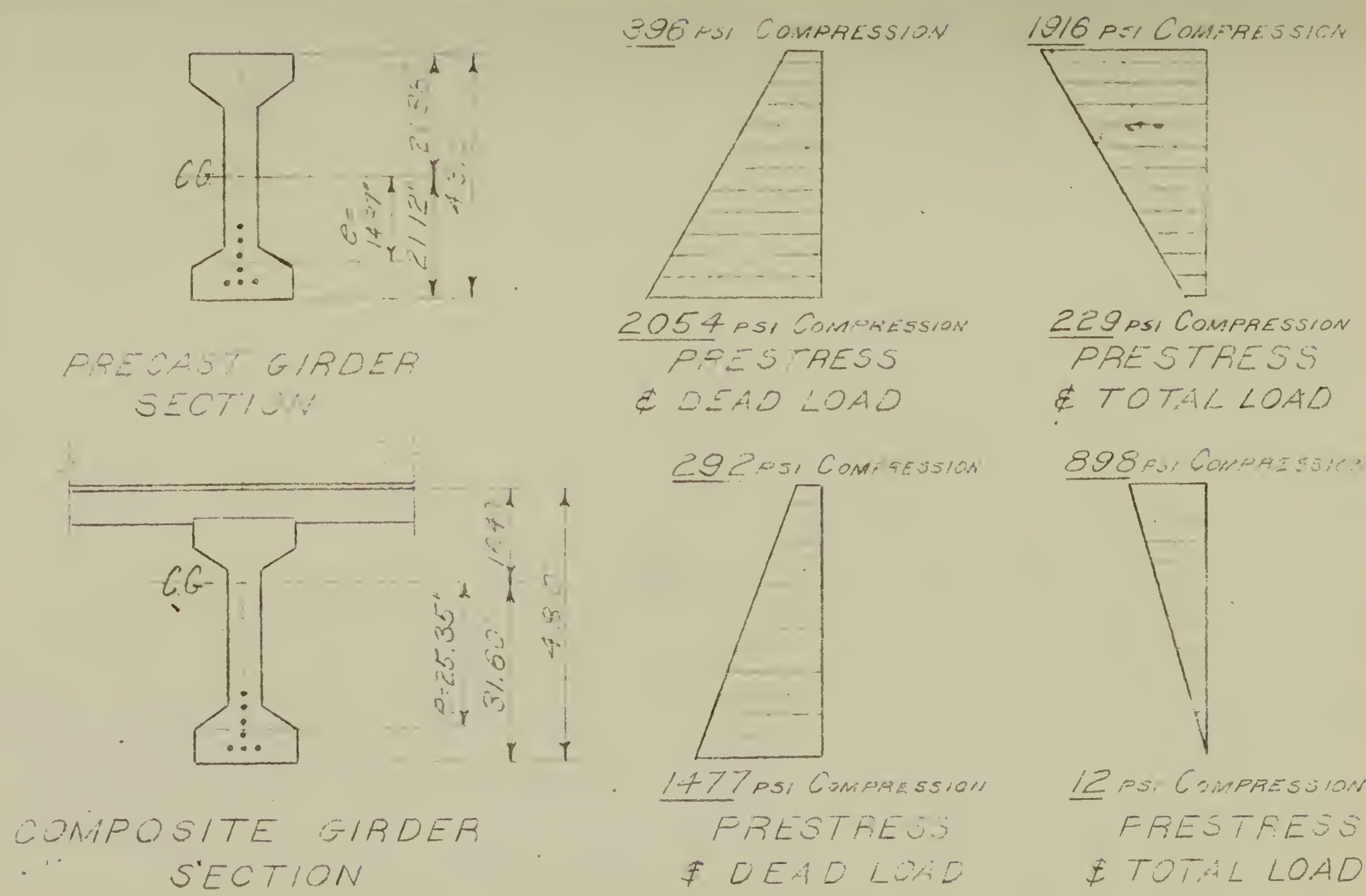

FIGURE NO. 10 STRESS CONDITIONS IN THE 30' GIRDER

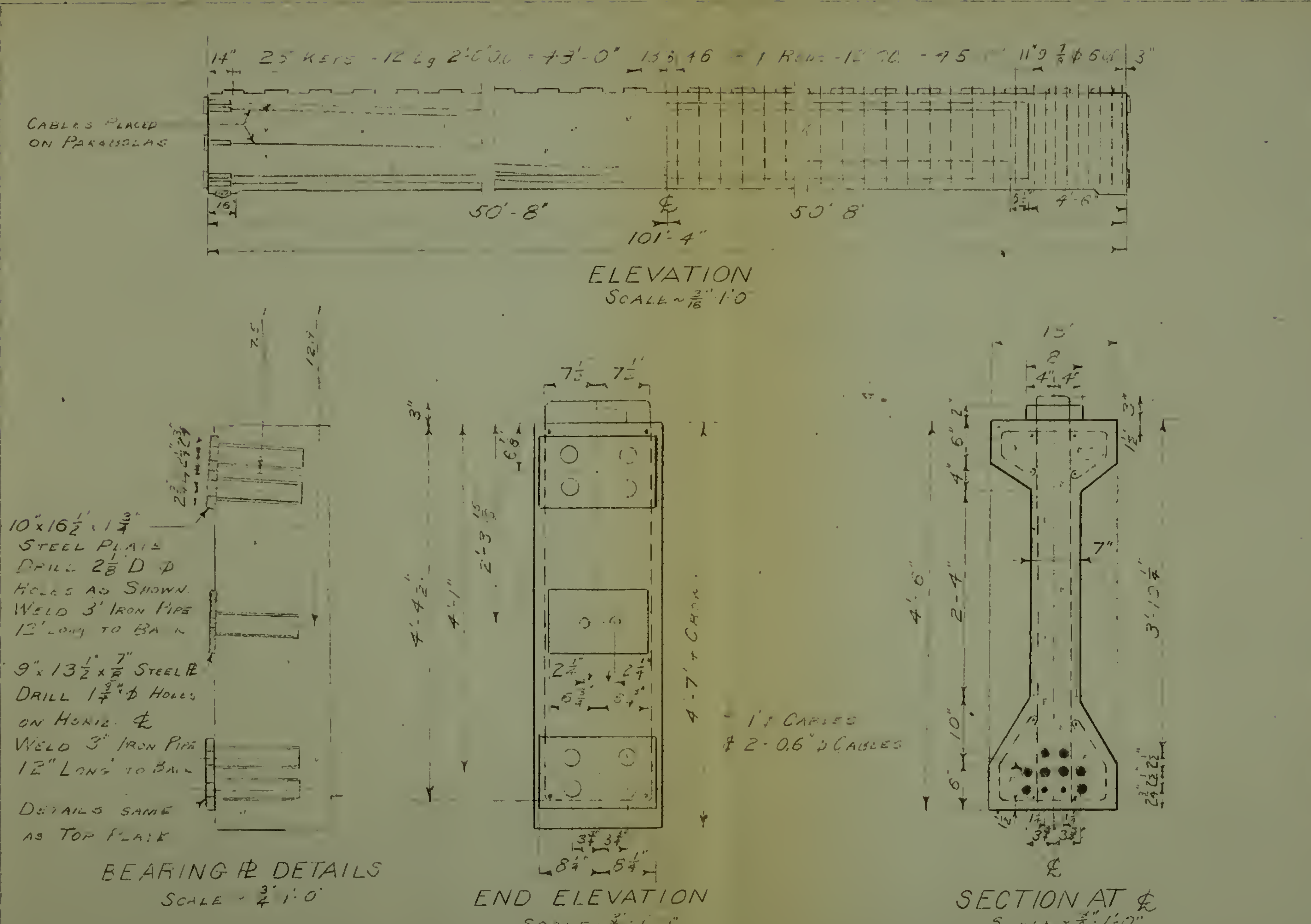

FIGURE No 11 DETAILS OF 100' GIRDER

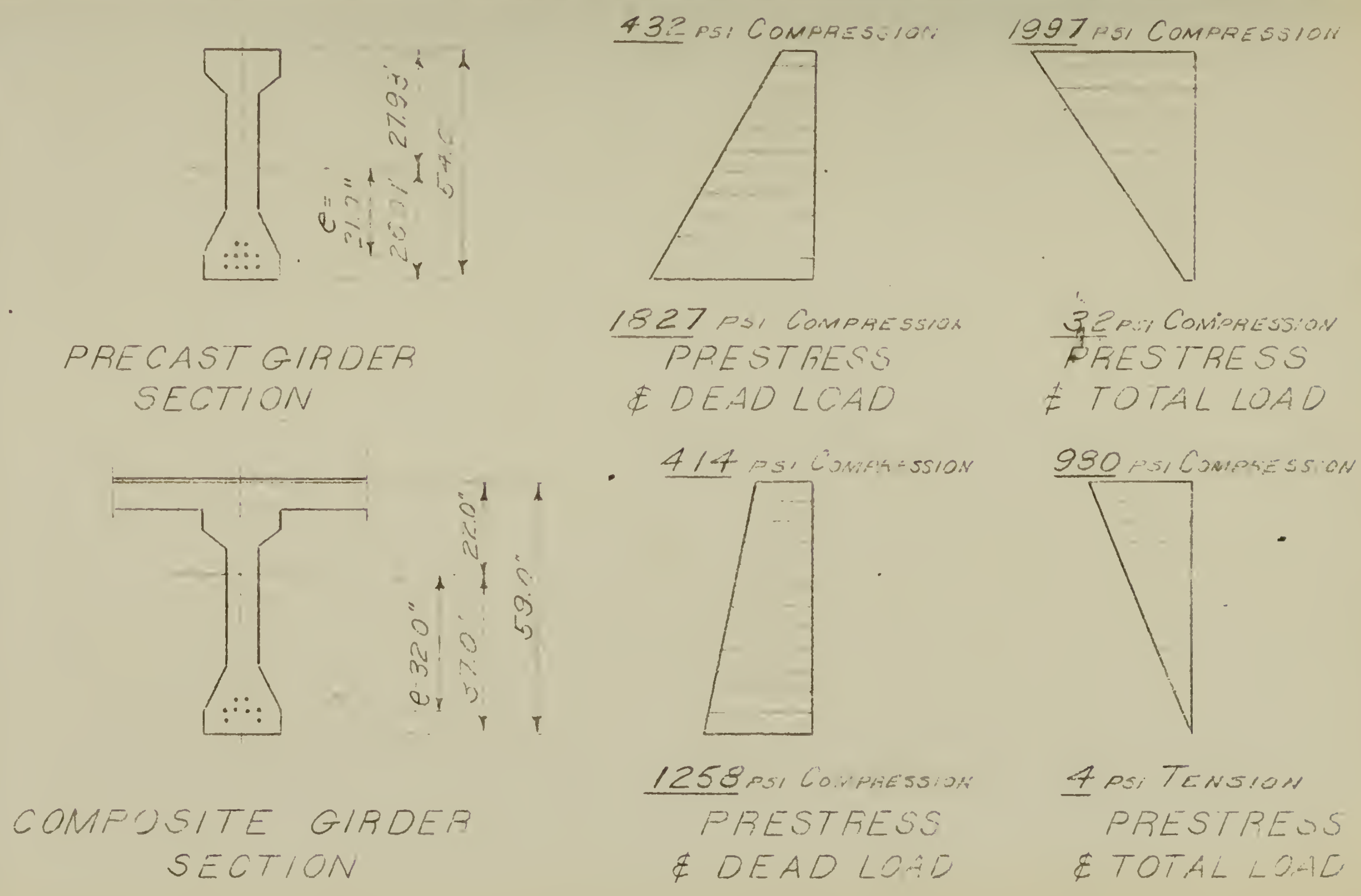

FIGURE NO. 12 STRESS CONDITIONS IN THE 100' GIRDER

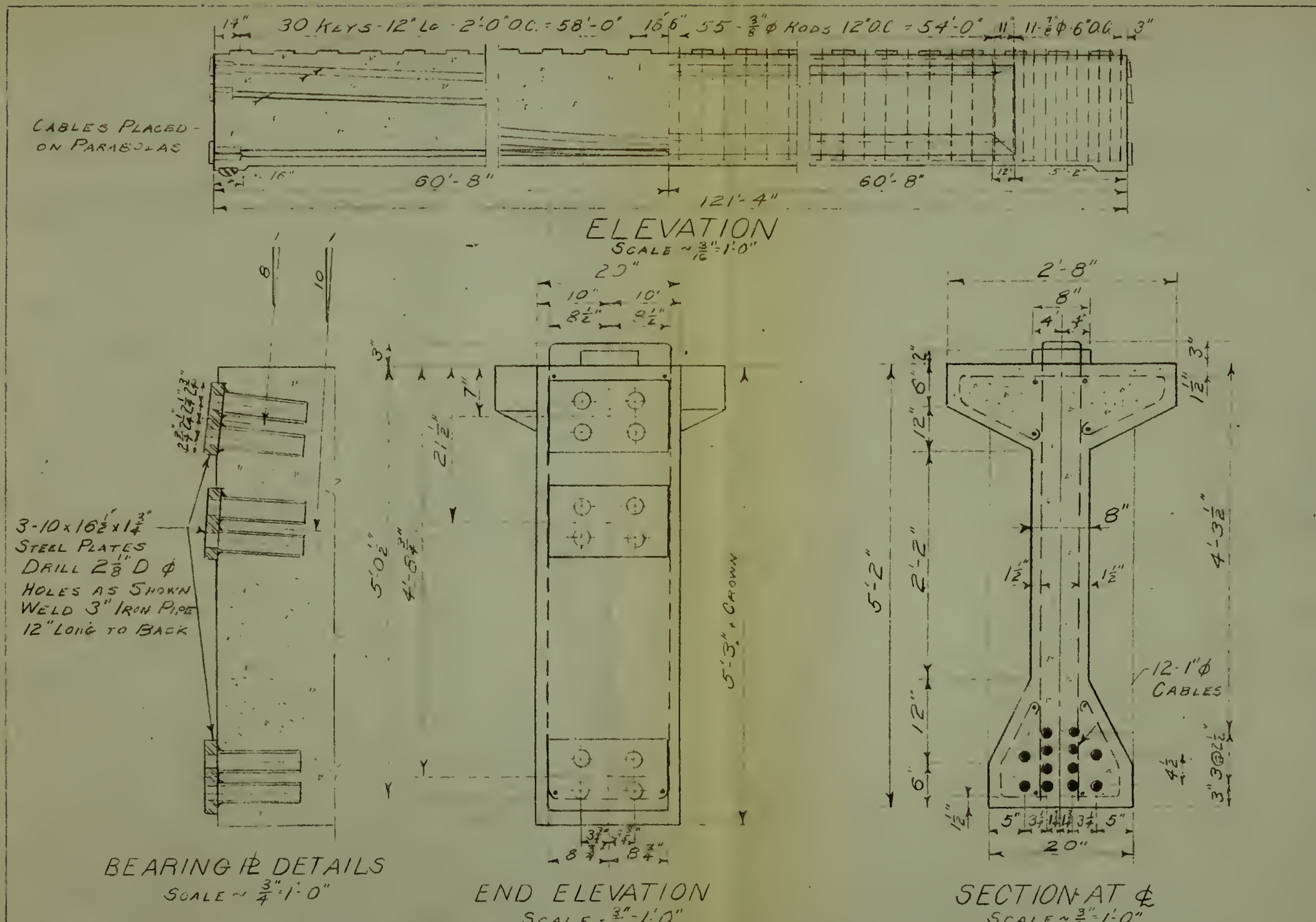

FIGURE No. 13 DETAILS OF 120' GIRDER

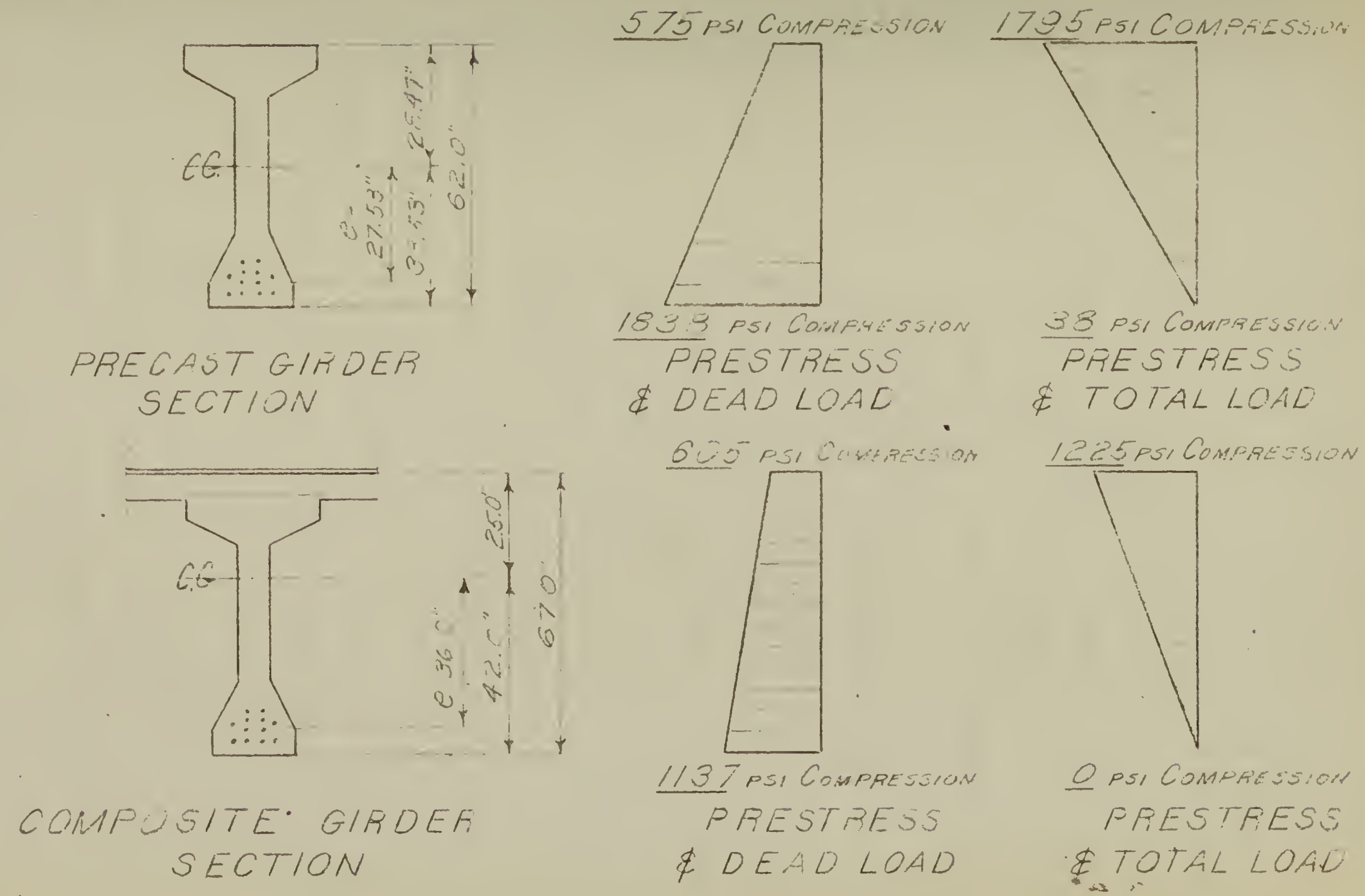

FIGURE NO. 14 STRESS CONDITIONS IN THE 120' GIRDER

C. Conclusions

After considerable experimentation with sections for the 40' span girder the authors decided on a cross section very similar to the final one shown in Figure 5. The shape seemed logical and is easily explained by the necessity of balancing up the composite section so that it wouldn't be overtopped, so, to speak, for balanced stresses. Shear or principal tensile stresses were large, but since other section trials failed for one or more of the condition equations this design approach was used for the sixty and eighty foot span designs. The design of the eighty foot span girder by this method led to excessive bending stresses and impossible shear stresses. A trial of a symmetrical section surprizingly corrected the difficulties. On this discovery the authors reviewed the previous two designs (40 and 60 foot span bridges) and ran new trials under the impression their previous theory had been inaccurate. The new girder sections however were not workable. Utter confusion reigned for a brief period until an overall survey of dead and live load moment ratios was made for the various span lengths. As the result of the survey the trend was clearly indicated. The ratio of live load moments (girder Ma to composite section Ma) clearly shows the transition from a large bottom flange to a symmetrical section to a large top flange in order to arrive at a balanced stress condition which meet economically the criteria established. The moment ratios are established from paragraph B - 3 above and are as

follows: 40 ft. span - 1:1.84; 60 ft. span - 1:1.35; 80 ft. span - 1:1.10; 100 ft. span - 1:0.90; 120 ft span - 1: 0.72. The discovery of the live load ratio indicator confirmed the original theory and explained the trial results for the eighty foot span bridge. Additional revision to the girder sections for the forty and sixty foot bridges led to improved shear conditions in both beams and the cross-sections for the 100 foot and 120 foot span bridges were obtained after several trials.

An interesting point was brought to light concerning prestressed girder bridges designed with the deck slab considered as not contributing to the cross-section properties. Some bonding is certain to occur between the girder and the deck slab. When such bonding occurs the center of gravity of the section will shift upward to some unknown point and the stress conditions due to the changed eccentricity of the cable will be materially altered. It was noted in the authors designs that the position of the center of gravity of the cross-section was very important, and it moves upward during the curing of the slab to its new position in the composite section. A variation of the eccentricity by a few inches could place the extreme fibers of the girder in tension.

VI FOUNDATIONS AND PIERS

Each of the precast, prestressed girder bridges have been designed as standard modular units to be constructed at any advanced base and not for any particular site or foundation conditions. The end abutments and the intermediate piers will require separate design that will be dependent upon the foundation conditions encountered at the bridge site. It is envisioned that the end abutments would be of the type employed for ordinary concrete deck girder bridges with breast wall, wingwalls and footings, all of reinforced concrete construction. It is planned that the intermediate piers would be simple pile bents with concrete cap or bridge seat for simplicity and to reduce the construction time to a mimimum.

The precast, prestressed method of construction will require a slightly wider bridge seat or bent cap than the normal construction in order to permit pouring of a protective concrete cover over the prestress steel end fittings and in order to provide proper distribution of stresses within the end block. This requirement is shown in Figures 2, 5, 7, 9, 11, and 13. The protective concrete cover may be poured simultaneously with the bridge deck slab and and diaphrams.

One advantage of the prestressed design is that the girder section is approximately one half that required for ordinary concrete design. This results is a smaller dead load for the bridge structure and, therefore, will permit a

[illegible]

saving in foundation and pier costs. The prestressed girders are also more shallow than the ordinary concrete girders thereby providing greater clearance or requiring less structure height.

VII BRIDGE DECK DESIGN

The roadway width, overall width of the bridge and deck slab were standardized for all the bridges of the series to facilitate development of package units or components for the advanced base construction. The standard bridge slab, curb, and guard rail are shown in Figure 4. The clear roadway width of twenty-six feet was selected for the two lane bridge resulting in an overall bridge width of thirty-two feet. The dimensions and clearances meet the requirements of the A.A.S.H.O. Specifications.

The roadway slab or deck, which has been assummed to act as the top flange of the composite girder in the manner of Tee beam design for ordinary reinforced concrete, has been designed for concrete with an ultimate compressive strength of 3750 psi. The design was accomplished in accordance with Paragraph 3.3.2 of the A.A.S.H.O.Standard Specifications for the H20 - S16 - 44 loading. The floor slab was assumed to act as a continuous transverse structure. A summary of the deck slab design is furnished as follows:

Clear Span: 4 ft. -0 inches

Distribution of Wheel Loads:

$$E = .6S + 2.5 = 4.9 \text{ feet}$$

Maximum Positive and Negative moments

$$M = \pm \frac{.2PS}{E} = 31,400 \text{ inch pounds}$$

IMpact Load:

$$I = .4 \times 31,400 = 12,500 \text{ inch pounds}$$

Dead Load Moment:

$$M_D = .8\ (1/8\ w\ l^2) = 1,680 \text{ inch pounds}$$

Total Design Moment:

$$M_t = 45,580 \text{ inch pounds.}$$

Required Depth of Slab:

Depth to reinforcing steel	= 4.75 inches
Cover	= 1.25 inches
Wearing Surface	= 1.00 inches
Total	= 7.00 inches

Reinforcing Steel:

A_s = 0.54 square inches per foot of slab.

Use 5/8 inch round bars at 6 inches center to center as positive steel.

Use 5/8 inch round bars at 12 inches center to center as the top slab reinforcement and bend up every other bottom steel bar at the supports to provide 5/8 inch round bars at 6 inches center to center negative steel, The bent up bars resist the slab shear adjacent to the supporting longitudinal girders. For longitudinal temperature steel use 3/8 inch round bars at 12 inches center to center both top and bottom.

The bridge guard rail was not designed in detail since standard designs are readily available; and, further, the design proceedure is well known to practising engineers.

VIII CONTINUOUS PRESTRESSED BRIDGES

The primary objective of this thesis is to develop a series of precast, prestressed girder bridges that are economical to build and which meet the many special requirements for modular advanced base construction. This plan envisions standard reusable forms, and packaged units or components of the necessary prestress steel, ordinary reinforcing steel and cement. The girders and cross- sections of the deck girder bridges have been designed with these objectives as the principal requirements. It was also desired to utlize the same standard girders for continuous prestressed bridge structures.

The theoretical approach employed by the authors in investigating the application of the standard girders to continuous prestressed bridges is that presented by Mr A. L. Parme and G. H. Paris of the Portland Cement Association in their paper entitled, "Analysis of Continuous Prestressed Concrete Structures."[8]

There is a basic difference between simply supported prestressed members and continuous prestressed members. As shown in Part II of this thesis, the moment induced in a simply supported member by the prestress force is directly

8 Proceedings of the First U. S. Conference on Prestressed Concrete, August 1951, Massachusetts Institute of Technology.

proportional to the eccentricity of the cables with respect to the centroidal axis of the member. This is not generally tme for the continuous prestressed member since the tension in the cables creates elastic deformations in the member which are resisted by the restraint at the supports. The support restraint thereby induces moments in the member which must be superimposed on the moments produced by the eccentric prestressed force.

The moment produced by the prestress force may be determined by considering the entire member as a free body, Figure 16 (a). The curved parabolic cable exerts a horizontal force P and a vertical force P tan ϕ at both ends of the member. The vertical force can be neglected since it is applied over the support. The curved cable also exerts a uniform normal force on the concrete which is equal to the prestress force, P, divided by the radius of curvature of the cable. For equilibrium of the free body the sum of the vertical forces equals zero or:

$$w_p \, a \, L = - P \tan \phi$$

where a is a coefficient indicating a ratio of the length. The slope of the parabola is equal to $\frac{2\,b\,x}{(a\,L)^2}$; therefore at a point x = a L,

$$w_p = - \frac{2Pb}{(aL)^2}$$

and when a = $\frac{1}{2}$ and b = C

$$w_p = \frac{-8Pc}{L^2}$$

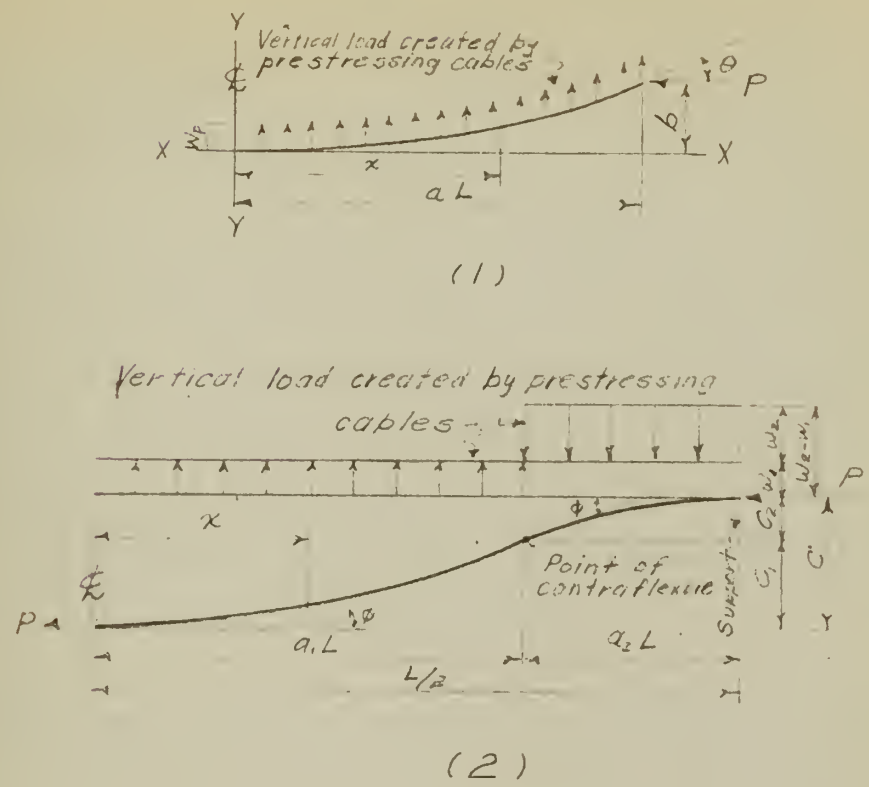

FIGURE - 16 A

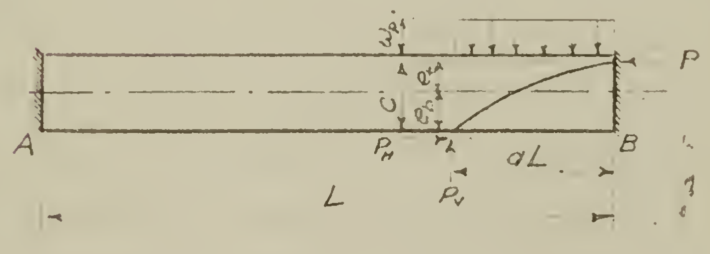

FIGURE - 16 B

The prestress moment then at any section, x, for equilibrium

$$M_p = P(c + e_1) + \frac{w_p L^2}{8}\left[1 - \left(\frac{2x}{L}\right)^2\right]$$

and substituting wp,

$$M_p = P\left[e_1 + c\left(\frac{2x}{L}\right)^2\right]$$

Thus the prestressed member may be analyzed by expressing or representing the effect of the prestress force by equivalent external loads. The problem is reduced to determing moments produced by uniform loads on portions of the structure. The final moments can be readily calculated by the method of moment distribution, since tables for fixed and moments due to partial uniform loads are available.

The above derivation applies to continuous curved cables and it is oftentimes desireable to employ discontinuous cables adjacent to the supports to effectively resist the negative moment. The use of discontinuous cables permits precasting of the girders for the individual spans of the total continuous structure. The girders are precast as simply supported prestressed beams, and after being placed are prestressed to resist the negative moment by means of short cables curved over the supports. This is considered to be the more economical method of construction, eliminating the need for extensive centering, shoring or cribbing to support the forms.

The analysis of the prestressed member with discontinuous curved cables is similar to that previously described

for members with continuous curved cables. The forces considered acting on a member with a curved cable at one end are shown in Figure 16 (b). The end moments induced by P_V and w_p are:

$$M^F_{BA} = -\frac{a^2 (6 - 8a + 3a^2)\, w_p L^2}{12} + a (1 - a)^2 L P_V$$

$$M^F_{AB} = -\frac{a^3 (4 - 3a)\, w_p L^2}{12} + a^2 (1 - a) L P_V$$

The end moments caused by the horizontal force, P_H, are:

$$M^F_{BA} = - P_H c_b (1 - 3a)(1 - a)$$

$$M^F_{AB} = P_H c_b a (3a - 2)$$

Since:

$$w_p = \frac{2 Pc}{(aL)^2}$$

$$P_H \approx P \text{ approximately}$$

$$P_V = \frac{2Pc}{aL}$$

$$c_b + c_x = C$$

By addition and substitution:

$$M^F_{AB} = Pc_t\, a \left[\left(1 + \frac{c_b}{c_t}\right)\left(\frac{4}{3} - \frac{3a}{2}\right) + \frac{c_b}{c_t}\left(3a - 2\right)\right]$$

$$M^F_{BA} = Pc_t \left[\left(1 + \frac{c_b}{c_t}\right)\left(1 - \frac{8a}{3} + \frac{3a^2}{2}\right) - \frac{c_b}{c_t} (1 - 3a)(1 - a)\right]$$

When the curved cables are symmetrical about the centerline of the member the equation for computing the fixed end moment

resulting from discontinuous curved cables is:

$$M^F_{AB} = M^F_{BA} = P e_s \left[\left(1 + \frac{c_b}{c_t}\right)\left(1 - \frac{4a}{3}\right) - \frac{c_b}{c_t}\,(1 - 2a) \right]$$

The final moments in the structure can be computed by moment distribution after the fixed end moments are obtained. The moment at any section along the member due to the prestress forces may be obtained by combining the final end moment algebraicly with the moment or moments produced by the prestress force or forces at the particular section.

The approach or procedure of the design is merely an extension of that presented in Parts II and III of this paper, with certain additional condition equations and points to be investigated. First, the maximum positive and negative moments at the supports and at the midspan points are determined by moment distribution for the live load plus impact and the added load of the superstructure above the girders. Secondly, the girder section is proportioned and the prestress forces determined as outline in Part II. The effect of the prestress force or forces upon the final moments is then computed and the girder is redesigned accordingly. Certain additional conditions must then then be investigated for the continuous structure.

The additional conditions which become important for a continuous linear prestressed member are temperature effect, settlement of supports, and the section of the member where

the moments due to the prestress forces cancel each other or are equal to zero. For members with continuous curved cables or combinations of discontinuous cables there occurs one or two sections at which the moment due to the prestress force is zero. This is true because the eccentricity is zero.or because the several cables acting through several eccentricities cancel out any prestress bending moments induced by the other cables. This particular point of the member should coincide with the point of inflection or zero moment point of the combined dead and live load moment curve or curves. The member cross-section at the zero moment point may then be proportioned or checked to resist only the bending moment resulting from the prestress force or forces and shear at the section.

The continuous prestressed member must be investigated very carefully for the end shear and resulting principal tensile stress. The axial compressive stress exists as for the simply supported beam, but the prestress cables do not have the same effect in reducing the end shear due to dead and live loads. Along the length of the continuous beam where the cable is curved parabolically upward to resist the positive moment, a negative shear force exists that opposes the positive live and dead load shear as for the simply supported member. However, along the length of the continuous beam where the cable is curved downward to resist the negative moment, the cable effect adds to the positive shear

produced by dead and live loads.

A practical design condideration presented by Professor G. Magnel [9], concerns the limitation on the number of spans of a continuous prestressed structure with continuous curved cables. Frictional resistance developed between the cables and the concrete at the time of post-tensioning causes a loss of some tension in the cables. As the result of experiments, Professor Magnel recommends that not more than three spans be used, and he recommends that the cables be initially overprestressed by approximately 5% and then reduced to the proper tension before sealing the cable ends.

If the deck slab is assumed to act as an integral part of the composite continuous girders, then a second set of design conditions must be satisfied in a manner similar to that described in Part II. The deck slab will materially assist in resisting positive bending moments, at the midspan section, but will not materially increase the ability of the concrete girder section at the supports to resist negative bending moments. The integral or contiguous action of the cock slab acting as a portion of the composite girder therefore is less effective for a continuous member than for a simply supported member.

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9. Prestressed Concrete, by Gutave Magnel, Second Edition, 1950, Concrete Publications Limited, London

Preliminary investigations were made concerning the application or adaptation of the standard modular girders described in Parts IV and V to continuous girder bridges. Detailed investigations for all types of construction procedure and span length combinations for continuous structures could not be accomplished in the time available for preparation of this thesis. It is considered that the complete design of a single continuous girder bridge is such a lengthy process of trial and error calculations combined with judgement and experience that it might well be the subject of a complete thesis.

The preliminary investigation of the application of the standard girders to continuous bridge construction at advanced bases revealed a number of practical and theoretical reasons why the standard girders are not suitable for this adaptation as follows:

1. In order to take full advantage of continuity, the girders should be designed to carry the added load of construction of the slab and superstructure as well as the live load. Using the precast method of construction the girders should be proportioned approximately symmetrical about the centroidal axis to effectively resist the positive and negative moments imposed. The standard girders presented in Parts IV and V are unsymmetrical in cross-section since they were designed

1. for maximum economy under a particular combination of positive bending moments, and therefore the standard girders do not readily lend themselves to adaptation to continuous structures. The feature of supporting the deck slab by the girders while pouring concrete becomes very important and critical for spans over eighty feet in length, since this construction load moment actually exceeds the live load moment.

2. The most important reason for not adapting the precast modular prestressed girders for continuous structures by the use of discontinuous curved cables at the supports is the additional time required for construction. The precast method is of value only if the girders carry the construction and dead load of the slab and superstructure; since if the forms are supported separately for the slab, the girders might just as well be poured simultaneously with the slab, requiring only a small additional quantity of cribbing and centering. Using precast girders and short discontinuous cables

over the supports, it is necessary to pour a section of concrete girder joining the precast girder ends prior to prestressing the cables. The prestressing operation should not be undertaken until the concrete reaches its ultimate design strength, approximately 30 days. Thus, adaptation for continuity would result in a project completion date delay of 30 days over the method employing simply supported members. Such a period of delay is not considered permissible for emergency advanced base construction.

3. The standard girders were designed with mimimum web thickness for maximum economy. There is not sufficient space or clearance in the web for installation of sleeves or pipes for the discontinuous curved cables that are required to resist the negative moments at the supports. In order to accommodate these additional cables it would be necessary to considerably alter the standard steel forms to provide a thicker web for the length of beams adjacent to the supports.

4. The problem of interference of the short

discontinuous cables and the end bearing plates of the full length cables must be solved. This results in additional steps in the design procedure: adjustment of eccentricities to prevent coincidental axes; and allowance of space for end fittings.

5. The short discontinuous cables also require bearing plates and end fittings where they emerge from the bottom fiber of the girder. The girder would have to be strengthened at this section to better distribute the concentrated vertical and horizontal loads in a manner similar to the end block construction. This intermediate stiffner block would necessitate extensive alterations to the standard steel forms for the modular girders.

6. For economy of construction, the choice of a common cross-section is indicated for the full length of the continuous girder. Such a procedure then would suggest investigation of various span lengths to obtain nearly equal maximum positive and negative moments, providing foundation conditions and the economical span length considerations would

permit such a variation of span lengths.

7. A further comparison that should be made for span lengths of eighty to one hundred and twenty feet is the true economy of precast method of obtaining continuity compared to the simple spans. The continuity generally will result in a somewhat smaller cross-section of the girder, but involves the additional expense of the short discontinuous cables and of the pouring the concrete section necessary to join the simple girder ends. A majority of the saving in section for an I shaped girder will be in reduced depth of the web. For a thin web member this does not result in the saving of a very great quantity of concrete.

In summary, it is the opinion of the authors that it would not be economical or practicable to adapt the standard modular prestressed girders to the construction of continuous structures at advanced bases for the reasons previously enumerated. It is considered that it would be necessary to design girders for a particular continuous bridge structure having defined span lengths, including approach spans. From the preliminary investigations undertaken, it was revealed that the design of a complete and detailed continuous prestressed

bridge structure would be a project of considerable magnitude, exceeding the time available for preparation of this thesis. These initial investigations did reveal that the design of continuous bridges with span lengths greater than 120 feet should result in greater economy of construction. Due to the additional web width required to contain the continuously curved or discontinuous curved cables for longer spans and because of the resulting greater weight of the precast beams, the authors are of the opinion that a cast-in-place box section girder would be more suitable and economical for span lengths in excess of 120 feet. Further, the top flange of the box section may be considered to act as the bridge deck slab and be designed accordingly. Such a section would be designed to utilize continuity over intermediate supports.

IX CONSTRUCTION PROCEDURES

The construction procedures for the prestressed precast modular girder bridges have been partially determined by the design criteria presented in Section II of this paper. The construction of the bridges at advanced bases further limits the construction methods that may be employed.

The structures have been so designed that the construction equipment normally available to construction forces at the advanced base will be adequate to accomplish the bridge projects. The concrete aggregate crushers, rolls and screens utilized for other advanced base concrete projects are capable of producing the control necessary for the high strength concrete. The "Table of Allowance" or "Table of Equipment" concrete weigh-batch units and mixers will also permit adequate control of the mix to produce the required 5,000 psi concrete for the precast girders. Concrete vibrators are normally available to the construction forces.

It is envisioned that the standard steel forms for the girders, other form material, prestress steel cables, reinforcing steel and cement will be prepared in standardized packages or components for each span length that has been designed. For larger bridges multiples of the standard span packages may be planned and ordered in advance of embarkation of the construction forces.

The basic concept of the designs presented has been the precast method of construction of the prestressed girders.

The advantages to be derived from the use of the standardized precast members in bridge construction are summarized as follows:

1. Savings in forms, falsework, and placing of concrete.
2. Elimination of a large amount of on-the-job labor.
3. Speed of construction.
4. Closer control of the concrete mix, placing, and curing, obtained through a mass production factory type operation and resulting in a better structure.
5. Precasting of the girders can be accomplished while the foundations, piers and abutments are being constructed.

Of course, particular care must be taken in placing the concrete in the steel forms for the thin section I and Tee members.

Temperature steel and end block stirrups could be packaged already bent and ready for placing. Accurate bending is desireable since the temperature stirrups have been designed as hairpins to align the cables perfectly and quickly and furnish tie supports. The standard steel forms mentioned above would be prepared with guides and indicators for the steel to reduce the skill involved in the steel placing operation.

It is planned that high early strength concrete would be used, permitting removal of the forms within four days after pouring. It is envisioned that a sufficient number of the standard steel forms, shown in Figure 2, would be available to permit pouring of several of the girders at one time. A group of several girders could then be poured every four days by re-use of the standard forms, thus obtaining a true production line method of construction. If greater speed of construction is desired, steam boxes may be employed to provide the optimum curing temperature of 135° F and by this method the forms could be re-used every second day. The standard steel forms are designed with bottom hinged, shaped and stiffened side members. The hinged sides are connected to cross angles that extend under the bottom plywood part of the forms, permitting removal of the steel side sections without disturbing the concrete girder. Diagonal steel braces with adjusting turnbuckles provide the required rigidity for the steel sides and permit adjustment to true position and alignment.

Prestressing of the cables may be accomplished as soon as the concrete reaches its design ultimate compressive strength. When high early strength cement is used, the cables may be prestressed after some twenty days curing time. If steam curing is employed, the cables may be prestressed within three or four days after pouring. The use of the "Roebling Type" standard cables greatly reduces the time required for the prestressing operation.

The precast modular girders are provided with lifting U bolts or special stirrups at each end block for handling of the members. The girders are designed for a two point lift. The girders for the 40, 60, and 80 foot spans are light enough in weight to be lifted by a single crawler crane employing a spreader beam for the two point pick. The girders for the 100 foot span bridge require two fifteen ton capacity cranes for handling, each crane lifting one end of the girder. The girders for the 120 foot span bridge necessitate the use of two twenty-five ton capacity cranes for lifting and placing. Cranes of fifteen and twenty-five ton capacity are normally available at the advanced base as this capacity crane is often required for other types of construction projects.

After the precast girders have been placed on the piers or abutments, the precast diaphrams or spreaders are placed into position. These precast diaphrams are grouted at the ends to assure a firm bearing and then stranded cables are installed to prestress the structure transversely. This prestressing operation assures proper placement and combined action of the girders and diaphrams. The diaphrams were designed with temperature steel only, depending upon the transverse prestressing to 25% of the ultimate strength of the concrete to prevent tension cracks in the concrete.

As provided in the design criteria, the prestressed girders are designed to support the dead load of the deck

slab, forms, and a construction load. A typical arrangement of the forms supporting the floor slab concrete is shown in Figures 2 and 4. The forms are designed so that standard plywood sections are supported by wales and braces attached to the bridge girders. These plywood panels and timber supporting members may be reused as the construction progresses along the length of the bridge. The curb section may be poured at the same time as the floor slab. The guard rail may be precast totally or in sections, depending upon the length of the bridge, and secured to the curb section. This method of construction permits use of the bridge as soon as the deck slab has been cured.

The construction procedures and methods outlined above are considered to permit construction of the bridge in the minimum length of time and to result in more economical construction as compared to other methods.

X SUMMARY AND CONCLUSIONS

The design of a series of precast prestressed concrete bridges for advanced base construction has been presented in this thesis. Deck girder type concrete bridges composed of mudular prestressed girders of 40, 60, 80, 100 and 120 foot spans have been conceived, in so far as possible, to reduce the materials and construction time to a minimum, consistent with good design practice. The designs have been slanted toward mass production precast methods of construction.

The modular girders have been proportioned in section to obtain an efficient use of the concrete and the high strength steel for prestressing, in accordance with the varying dead load to live load ratios for the different span lengths and for the several construction phases. The method of approach is based on the assumption that the bridge deck slab acts as a contiguous or integral portion of the prestressed Tee shaped girders to resist the live load plus impact for the H 20 - S 16 - 44 loading conditions.

Although the designs have been adapted for Naval advanced base construction, they are considered to be equally applicable to state or local projects. The advanced base requirements result in greater construction speed and greater economy of materials and labor, and such attributes are desired for any construction project. While the modular prestressed girders have been designed for bridge construction,

it is believed that they are adaptable, with only slight modification, to pier and wharf construction. The use of such girders for pier and wharf construction furnishes the added advantage of long life due to the protection against corrosion by salt water afforded by the compressed concrete due to the prestressing.

The construction method of precasting the prestressed modular girders for the bridges results in numerous advantages as follows:

1. Savings in forms, falsework, and placing of the concrete.
2. Permits re-use of standardized forms for the girders.
3. Elimination of a large amount of on-the-job labor.
4. Speed of construction.
5. The girders can be completed and ready for placing by the time the piers, abutments and foundations are finished.
6. Eliminates the requirement for falsework, centering and cribbing for the deck slab forms.

The prestressed deck girder bridges have been so designed as to permit construction with advanced base equipment normally available to construction forces as "Table of Allowance Equipment." The bridges have been so planned that package

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[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

units or components can easily be prepared for a single span length, and a particular bridge would be composed of multiples of the standard component. Each component would contain the necessary standard forms, prestress steel, reinforcing steel and cement for a single span length, depending upon the feasibility of development of the local resources for concrete aggregate. Specially trained crews would complete the construction in a minimum of time.

The girders and the composite girders for the deck girder bridges are considered to possess all the attributes normally attained in prestressed concrete construction. These advantages are summarized as follows:

1. Prestressing makes concrete crackless which results in greater durability under severe conditions of exposure.
2. Prestressing makes it possible to use efficiently higher strength and higher quality concrete.
3. Prestressing makes possible the most efficient distribution of material for the member cross-section, such as I, Tee, and box-sections.
4. Prestressing minimizes deflections and reduces the depths of beams, girders and slabs, thus affording greater under clearance.
5. Prestressing results in maximum rigidity under design loads and maximum flexibility

[illegible]

[illegible]

[illegible]

[illegible]

[illegible]

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[illegible]

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under excessive overloads to give ample warning of impending failure.

6. The lighter weight structures have the attendant advantage of smaller and more economical foundations.

One of the most important conclusions derived from the design of this series of bridges concerns the shape and cross-section of the individual modular girders. A basic requirement is that the cross sectional area and section modulus be held to a minimum for greater economy. The resulting I sections are slender, well formed, and proportioned to the requirements of construction load moments and the live load moments. For the forty and sixty foot span girders the dead load moment of the superstructure and construction load moment are less than the live load moment, resulting in a girder with a larger bottom flange and a composite girder of Tee section. For the 120 foot span girder the dead load moment of the superstructure and construction load moment together exceed the live load moment. The girder and composite girder are both Tee shaped in section to obtain maximum economy of materials. There is a gradual transition for the girder section from inverted Tee for the forty foot span, to an I section, and finally to a Tee section for the 120 foot span girder. Economy of materials is re-emphasized by the use of the high strength 5000 psi concrete only for the basic girder. The deck slab,

which forms part of the top flange of the composite girder, is constructed of concrete having an ultimate compressive strength of 3,750 psi. For comparison, assume that the concrete prestressed girders are to be replaced with rolled structural steel shapes. It would require a $2\frac{1}{2}$ ton rolled steel section to replace the 5 ton prestressed girder for the forty foot span and nearly an 8 ton steel beam to replace the 13 ton concrete girder for the sixty foot bridge. Stated differently, concrete is serving with approximately half the weight strength in flexure of the rolled steel shapes.

Another comparison as to economy of materials can be made to ordinary reinforced concrete. The prestressed girder results in the saving of approximately 50% of the concrete and 70% of the steel required for an ordinary reinforced Tee beam for a deck girder bridge of the same span length. However, this is not considered to be a fair comparison, since the economical range of span lengths for an ordinary reinforced concrete deck girder bridge is generally agreed to be from 20 to 65 feet[10] while the economical range of span lengths for simple span prestressed girder bridges is considered by the authors to be 60 to 140 feet. Thus,

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10 "Design of Concrete Structures," by L.C. Urquhart and C. E. O'Rourke, Fourth Edition 1940, McGraw-Hill Co, New York, N. Y.

the prestressed girder bridge complements the ordinary reinforced concrete bridges, but does not necessarily replace them in the construction field.

Selection of the bridge span lengths is an economic problem determined largely by the particular site conditions. However, it is apparent that for the prestressed modular bridges proposed by the authors the economic advantage favors the longer spans, providing cranes of the required capacity are available to handle the heavier girders. Two 25 tons capacity cranes, largest of the standard advanced base crawler cranes, are capable of handling the largest girder of the series. For example, a single 120 foot span bridge would need no intermediate piers and would only require the placing of six girders weighing 54 tons each. Whereas, a 120 foot bridge composed of three forty foot spans would necessitate the placing of 18 girders weighing 5 tons each. The additional concrete required for the single span girders totals only ~~17%~~ 88% or approximately ~~19~~ 113 cubic yards of high strength concrete. It is believed the two intermediate piers would cost considerably more than the 19 cubic yards of concrete for the 120 foot span bridge. There are too many variable factors to draw any exacting conclusions from this data, but from the economy of materials viewpoint, it would seem that the longer span bridge would easily be justified.

Even for prestressed concrete there is a definite economic limit to span length, since with increasing span

lengths the ratio of dead load to total load moment increases rather rapidly. The authors are of the opinion that the range of economical span length for the precast prestressed girder bridges is approximately 60 feet to 140 feet.

The next logical step for longer spans is the design of continuous prestressed structures. Investigations revealed that the series of modular girders did not lend themselves to adaptation for continuous structures for theoretical, economical and practical reasons or considerations. It is considered that design for continuity in prestressed girder bridges would be practical and economical for span lengths of 150 feet and greater. Based on the experience gained in the design of this series of bridges, it is believed that a box-section girder would have desirable characteristics for the longer span continuous bridges, and the top flange could be designed as the structural deck of the bridge.

Very few continuous prestressed concrete bridges have been designed or constructed. This is a field or area that is believed to be of considerable importance to the construction industry and further investigations, designs and theses on this subject are recommended.

A further conclusion of importance concerns the design approach or theory. The assumption that the deck slab acts as an integral part of the composite beam or girder is

believed to be theoretically and practically correct. For this series of modular prestressed girders special provisions have been made to develop the necessary shearing strength at the junction of the deck slab and the simple modular girder. Even when no special provisions are made, some bond develops between the girder concrete and deck concrete and thereby a shearing resistance is introduced. This has the effect of changing the location of the centroidal axis of the girder and changing the eccentricity of the prestress cables, which may result in excessive tensile stress in the concrete if not properly considered in the design calculations. For example, the change in the center of gravity from the modular girder section to the composite girder section, after the deck concrete has cured and for the application of live loads, is a total of some 11.4 inches for the 40 foot span bridge with an equal change in eccentricity of the prestress cables. This change in centroidal axis is almost the same for all the girders of this series.

In final summary, the conclusions developed from this design thesis are listed in a more concise form:

1. The design criteria and specifications for the advanced base construction of the modular girder bridges have been satisfied with attendant economy of construction.
2. The numerous advantages of the precasting method of construction result in a better

and more economical structure.

3. The bridges as designed are capable of being constructed with the construction equipment normally available to construction forces at advanced bases.
4. The bridges are designed so that standard packages or components, consisting of the necessary standardized steel forms, prestress steel, reinforcing steel, cement and timber form material may be prepared in single span multiples mail order fashion and shipped prior to embarkation of the construction troops.
5. The prestressed modular girders and composite girders are considered to possess all of the advantages normally attributed to prestressed concrete construction.
6. The cross-section and shape of the modular girders and composite girders have been proportioned in accordance with requirements of the ratios of dead load to live load moments to achieve maximum economy of materials.
7. The range of economical span lengths for the simply supported prestressed girder

bridges as designed is considered to be 60 to 140 feet.

8. The longer spans of 80 through 120 feet in length are considered to be generally more economical, providing site conditions do not otherwise govern.

9. The precast modular girders for the span lengths of 40 through 120 feet are not considered to lend themselves to adaptation for continuous structures for theoretical, economical and practical reasons.

10. Further investigation, study and design of continuous girder bridges having span lengths greater than 150 feet is recommended.

11. The design assumption that the deck slab of the bridge acts as an integral or contiguous part of the composite Tee shaped girder is believed to be theoretically and practically correct.

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